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STORAGE RESERVOIRS

WITH EARTHEN DAMS

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INDIAN STORAGE RESERVOIRS WITH EARTHEN DAMS:

BEING

A PRACTICAL TREATISE ON THEIR DESIGN AND CONSTRUCTION

BY

WILLIAM LUMISDEN STRANGE,

UNDER-SECRETARY TO THE GOVERNMENT OF INDIA, PUBLIC WORKS DEPARTMENT;
MEMBER OF THE INSTITUTION OF CIVIL ENGINEERS.

"For in the wilderness shall waters break out, and streams in the desert. And the parched ground shall become a pool, and the thirsty land springs of water."—ISAIAH XXXV. 6, 7.

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PREFACE.

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It is probable that one of the first results of the inquiries made by the Indian Irrigation Commission will be the construction of storage reservoirs to serve parts of the country which are peculiarly liable to drought. Indeed, several such schemes have already been submitted, and investigations in respect to others—some of the largest magnitude—are now in progress. Seeing that but few works of this class have been constructed anywhere in recent years, and that they are now being commenced for the first time in certain provinces, it occurred to me that a book treating of the different problems which present themselves, both in the design and in the construction of storage reservoirs, would be of use as a guide to those entrusted with their execution. I have written the following pages, however, in the hope that they may prove helpful both to those who have already had to deal with this class of work as well as to those who have not yet had any connection with it.

My own experience has been gained in the Bombay Presidency, and I have endeavoured to describe the practice which usually obtains there, as well as certain modifications and improvements of it which appear to me to be desirable. Although that experience and practice are necessarily of a local character, it is not unlikely that similar conditions to those which prevail in Bombay will be found in other parts of the Empire which lend themselves to the construction of

storage reservoirs, and I therefore trust that this work will be of general utility in India.

Mr. Fanning observes[1]: "An earthwork embankment appears to the uninitiated the most simple of all engineering constructions, the one feature that demands least of educated judgment and experience." I hope that the following pages will show that this opinion of the uninitiated is entirely erroneous, and that for the proper design and construction of such a work a very considerable amount of skill and attention is absolutely necessary if success is to be attained.

The fact is that in the case of earthen embankments we are dealing with a material which is unsatisfactory and unreliable unless carefully treated, whereas, if it is properly utilised it has peculiar advantages of its own and is permanent to an eminent degree, as some of the oldest works in the world testify. Such embankments are, moreover, the cheapest structures whereby water can be stored, and they are particularly suitable for the employment of the large amount of unskilled labour which is available in ordinary times in India, and for which work has to be found in times of scarcity. I have therefore described in detail the precautions which it is most desirable to take—precautions which involve careful supervision rather than greatly increased expenditure. Naturally, they are needed more in the case of large and important works than in that of small and relatively insignificant ones.

Reservoirs in India are required either for irrigation or for the water-supply of towns. The former class, being the more important from an engineering point of view, has been described in detail, while the latter has been subsequently

[1] "A Treatise on Hydraulic and Water-Supply Engineering." 15th edition, 1902. Page 334.

dealt with at less length. The different parts of the complete project—the dam, the waste-weir, and the outlet—form the subjects of separate chapters, which, for the sake of easy reference, have been divided into numbered sections and paragraphs. The general conditions which have to be taken into account are discussed in the initial chapter, while the final one is devoted to miscellaneous subjects; at the end has been added a series of appendices which deal with and illustrate the general matters noticed in the earlier part.

The book is intended to be entirely practical, and, I believe, covers ground which has not been occupied before, as descriptive engineering works dealing with a variety of subjects do not usually go into complete detail in respect to any one of them. It has been my object to treat this one class of work as exhaustively as possible, so that an engineer not previously acquainted with it should, after studying these pages, be able to design correctly, and to construct securely, a storage reservoir with an earthen dam.

I am indebted to the Institution of Civil Engineers for permission to make free use of Paper No. 2996, which I contributed to Volume CXXXII. of their "Minutes of Proceedings," and of another Paper, which was not published, both of which deal with the same subject. I am also much obliged to Sir Thomas Higham, K.C.I.E., and to Mr. R. B. Buckley, C.S.I., for permission to make use of their books. I have acknowledged in the text my obligations to the other authorities whom I have consulted.

I have endeavoured to distinguish clearly between matters of ordinary practice accepted in Bombay and my own suggestions and recommendations, but, as the book is written impersonally, it may be as well, in order to avoid

any chance of misapprehension, to say that the following are put forward from the results of my own experience:—

	Para.
The substitution of subsidiary reservoirs for simple feed channels.	9
The crest wall of the dam	70
Breaching sections	75
The uniform construction of the dam . . .	80
The drainage of the puddle trench . .	96—98
The system recommended for the foundation and the drainage of the base of the dam	107, 108, 112, 113
The "Compound Dam"	128—130
The system to be adopted for the closure of a high dam	133
The waste-weir with parallel tail channel . .	181
The "Stepped Waste-Weir" for the utilisation of the flood-absorptive capacity of the reservoir	187—195
The recommendation for the adoption of the "Headwall on the centre line of the dam" to secure the subsidiary uses of the outlet . .	206
The sanitary treatment of catchment areas . .	227

The appendices, with the exception of Nos. 1, 6, 9, 23, 24, and 25, are original.

As it is generally found easier to follow a concrete example than an abstract description, I have, in order to make the latter clearer, taken for the former the project for the Máládevi Tank, which I drew up some years ago. This large scheme comprises practically all the features with which it is necessary to deal in the design and construction of a reservoir with a high earthen dam.

In conclusion, I may state for the benefit of engineers without Indian experience that the following are the

conditions which have to be met in India. The rainfall is almost entirely confined to the monsoon months, and is very capricious in amount and intensity. The rest of the year is generally characterised by a total absence of rain, and during this period a fall seldom occurs sufficient to produce replenishment. Constructional work is practically impossible during the rains, and the programme for the execution of the work has therefore to be confined to the seven fair-weather months of the year. Owing to these climatic conditions, and also to the tropical heat and storms, a large amount of storage has to be effected, and a large provision for the safe discharge of floods has to be made.

The labour available is unskilled, and the amount of skilled supervision is limited.

W. L. STRANGE.

SIMLA, *April*, 1903.

I HAVE to acknowledge the great assistance that I have received from Mr. G. W. Herdman, B.Sc., in the revision of the proofs, and from Mr. Buckley in arranging for the publication of the book.

W. L. S.

PRETORIA, *February*, 1904.

CORRIGENDA AND ADDENDA.

Page No.		FOR	READ
12	Fourth line from top	"be many times that"	"greatly exceed that"
24	Last figures in table at foot	0·36	0·32
63	2nd line below table	"paragraph 129"	"paragraph 128"
121	9th line from bottom	"running, say, a quarter of a ton to the foot, and"	"a quarter of a ton in weight"
136	6th line from bottom	"paragraph 207"	"paragraph 205"
157	Table—"Clear overfall weir—8 feet"	17·67	17·47
162	Paragraph 163, 5th line from bottom	"The mean width of weirs"	*Add*—"built of trapstone masonry"
202	10th line from bottom	"and the value"	"and the capitalised value"
203	13th line from top	"storage, at Rs.238"	"storage, capitalised at Rs.238"
203	Last sub-paragraph but one	Cancel.	
226	5th line from bottom	"paragraph 211"	"paragraph 201, p. 209"
229	Fig. 42	"Outlet piers to be at least 6″ thick"	"Outlet piers to be at least 8′ 0″ thick"
238	4th, 10th, and 11th lines from bottom	"rods"	"rod"
277	Appendix 8, column 1	7—9	7—8
278	Appendix 9, heading of columns	c.	c_2.
281	Appendix 11. *Add as Note* 5—"The larger co-efficients of discharge from a sharp-crested than from a broad-crested weir point to the desirability of forming a waste-weir wall with a sharp crest. This can be done by fixing an angle iron on the upstream side of the top of the wall, and by bevelling that top downstream. A considerable increase of storage or shortening of the weir can be thus effected."		
305	Paragraph 12, 8th line from top	"bed and sides"	"top and sides"
320	Appendix 19, Table 1, 2·1 feet	22·107	22·102
326	Form of Measurement Sheet	*Insert as Column* 7A—"Mean Cross-sectional Area."	
329	Appendix 21, Table IV., 41·7 feet	131·155	131·855
,,	Do., 43·7 feet	138·879	138·179
,,	Do., 80·9 feet	255·816	255·806
Plate	No. 4. Horizontal Scale	300 feet	600 feet
,,	No. 9. Note below heading	Appendix V.	Appendix XIII.
,,	No. 11. Figs. 19 and 20, Scale	$1\frac{1}{2}$″ = 1 foot	$\frac{3}{4}$″ = 1 foot.

TABLE OF CONTENTS.

CHAPTER I.

THE RESERVOIR.

I. *Selection of the Site.*

Para. No.	Subject.	Page No.
1.	Comparison of Near and Distant Reservoirs...	1
2.	Loss in Transit down Natural Stream Beds	2
3.	Loss in Transit down Canals	2
4.	Selection of the Site of the Dam	3
5.	Selection of the Storage Basin...	4

II. *The Catchment Area.*

6.	Classification of Tank Catchments	5
7.	Yield from the Catchment	6
8.	Partly Natural and Partly Artificial Catchments—Equivalent Catchment Area	7
9.	Subsidiary Reservoirs	9
10.	Wholly Artificial Catchments	10
11.	Measurement of Catchment Area	10

III. *Rainfall and Run-off.*

12.	Natural Disposal of the Rainfall	11
13.	Variations in Rainfall and determination of the Proportion of Run-off ...	11
14.	Necessity for maintaining several Rain-gauges	13
15.	Examples of the Proportion of Run-off to Rainfall	13
16.	Tabular Statement of Rainfall and Run-off	16
17.	The Effect of the Intensity of the Rainfall and the Condition of the Ground on the Run-off	16
18.	Tabular Statement of Daily Runs-off	17
19.	Estimation of the Run-off from the Daily Rainfall...	18

CHAPTER I. THE RESERVOIR—*continued.*

IV. *The Storage Capacity of Tanks.*

Para. No.	Subject.	Page No.
20.	Proportion of the Storage Capacity to the Yield from the Catchment ...	19
21.	Calculation of Storage Capacity	20
22.	Loss by Evaporation and Absorption	22
23.	Observations of Loss by Evaporation and Absorption in Tanks	23
24.	Observations of Loss by Evaporation and Absorption in Canals ...	25
25.	Comparative Cost of Storage	27
26.	Estimate of the Storage required for a certain Irrigable Area	27
27.	Estimate of Expenditure from Storage—Duration of Supply	28
28.	Preliminary Estimate of the Cost of a Storage Reservoir	28

V. *The Irrigational "Duty" of Water.*

29.	The Amount of Storage dependent upon the "Duty"	29
30.	The Proportion of Irrigable Area to Culturable Area under Command ...	30
31.	The Proportions of different Crops under Irrigation	31
32.	The Rotation of Crops	32

VI. *Tank Basins.*

33.	The Proportion of the Submerged Culturable Area to the Area Irrigable by the Project	33
34.	The Retentiveness of the Tank Basin...	34
35.	The Extent and the Utilisation of the Land to be Acquired	34

VII. *The Silting of Reservoir Beds.*

36.	The Necessity and the Methods for the Reduction of Silt Deposit ...	36
37.	The Classification of Silt	37
38.	The Proportion of Silt to Water	38
39.	The Determination by Cross Sections of the Amount of Silt Deposited...	39
40.	The Determination by Silt Boxes of the Amount of Silt Deposited ...	40
41.	The Determination by Observations of Inflow and Outflow of the Amount of Silt Deposited	40
42.	The Level at which the Outlet Sill should be Placed	41
43.	The Advantage of Earthen Compared with Masonry Dams in respect to the Silt Difficulty	42

VIII. *Survey and Investigation Work.*

44.	Survey for the Works	42
45.	Survey of the Capacity of the Tank Basin	43
46.	Rainfall and River Discharge Observations	44

IX. *The Utility of Tanks.*

47.	The Utility of Tanks in Times of Scarcity and Deficiency of Rainfall ...	45
48.	The Comparative Utility of Small and Large Tanks	46
49.	Revenue Prospects of Tanks	47
50.	The Suitability of Tank Works for the Employment of Famine Labour...	48

CHAPTER II.

THE DAM EMBANKMENT.

I. *Various kinds of Dams.*

Para. No.	Subject.	Page No.
51.	Classification of Dams	50
52.	Earthen Embankments...	50
53.	Advantages of Earthen Embankments	51
54.	Masonry Dams—Composite Dams	51
55.	American Types of Dams	52
56.	"Rock-fill" Dams	52
57.	"Hydraulic-fill" Dams	53
58.	Earthen Embankments with Masonry Core Walls	54

II. *Theoretical Considerations.*

59.	The Deficiency of the Theory of Earthwork	57
60.	General Causes affecting the Stability of Earthwork	58
61.	The "Historical Element" of Earthwork	58
62.	The Effect on Earthwork of Causes acting subsequently to its Construction	59
63.	The Behaviour of Puddling Clay with Water	61
64.	The Rate of Filtration through Soils—The Depth of the Puddle Trench	62

III. *The Design of Dam Embankments.*

65.	The Section of the Dam Embankment	63
66.	The Height of the Dam	64
67.	The "Free Board" of the Dam	64
68.	The Top-width of the Dam	65
69.	The Side-slopes of the Dam	66
70.	The Crest of the Dam—The Crest Wall	66
71.	The Width of the Dam at High-Flood Level and Full-Supply Level ...	68
72.	Allowance for Settlement	69
73.	The Sectional Area of a Dam Embankment...	70
74.	Sections Required at Particular Sites—Gorge Embankments—Dams on Inferior Foundations	71
75.	Breaching Sections	72
76.	Berms	73
77.	Toe Walls	74
78.	The Relative Cost of the Dam and the Waste-Weir	75

IV. *Materials for Dam Construction.*

79.	Selection of Material Necessary	76
80.	The Disposition of Material in the Section	77
81.	The Casings of the Dam	80
82.	The Utilisation of Spoil	80
83.	American Practice	81

CHAPTER II. THE DAM EMBANKMENT—*continued.*

V. *The Puddle Trench.*

Para. No.	Subject	Page No.
84.	The Object of the Puddle Trench	82
85.	The Position of the Puddle Trench	82
86.	The Depth of the Puddle Trench	83
87.	The Bottom-Width of the Puddle Trench	85
88.	The Side-Slopes of the Puddle Trench	86
89.	The Length of the Puddle Trench	86
90.	The Height of the Puddle Filling	87
91.	The Continuity of the Puddle Trench	87
92.	Filling the Puddle Trench	88
93.	Concrete Trenches	90
94.	Concrete Key Trenches	92
95.	The River Crossing of the Puddle Trench	92
96.	The Drainage of the Puddle Trench...	93
97.	The Construction of the Puddle Trench Drain	94
98.	Infiltration not Induced by the Puddle Trench Drain	97
99.	Puddle Trenches not Suitable for Famine Work	98
100.	Nulla Puddle Trenches	99
101.	Proposed Concrete Trench for the Bohio Dam, Panama Canal	99
102.	Excavation in the Reservoir Bed	101

VI. *The Puddle Wall.*

103.	Central Puddle Walls	102
104.	Slope Puddle Walls	103
105.	Puddle Walls not now Adopted in Indian Practice	104

VII. *The Drainage of the Dam.*

106.	The Necessity for Drainage	104
107.	The System of Drains Proposed—The "Surface Drain"—The "Down-stream Drain"	105
108.	"Rear Drains"...	108
109.	Open and Underground Drains	110
110.	The Self-drainage of Earthwork	111

VIII. *The Foundations of the Dam.*

111.	Order of Suitability of Natural Foundations	112
112.	"Foundation Trenches and Drains"	114
113.	Benched Foundations of the Dam	116
114.	Deep Black "Cotton-Soil" Foundations	117
115.	Sandy River Bed Foundations	117

IX. *The Construction of the Dam Superstructure.*

116.	General Formation	118
117.	Watering	118
118.	Mixing and Spreading...	119
119.	Consolidation	120
120.	Thickness of Layers	122

CHAPTER II. THE DAM EMBANKMENT—*continued.*

IX. *The Construction of the Dam Superstructure*—continued.

Para. No.	Subject.	Page No.
121.	Slopes of Layers	123
122.	Uniform Construction Necessary	123
123.	Junctions of Earthwork—Cross-sectional and Longitudinal	124
124.	Finishing off the Dam...	125
125.	Observations during Construction	126
126.	General Remarks: Setting out the Centre Line—Setting out Templates —Minor Arrangements—Method of Executing Work	127

X. *Compound Dams with Drystone Toes.*

127.	Objections to High Earthen Dams	129
128.	The Drystone Toes of the "Compound Dam"	130
129.	The Construction of the Drystone Toe	131
130.	The Advantages of the "Compound Dam"	133

XI. *The Closure of the Dam.*

131.	The Importance of the Closure	133
132.	The Ordinary Method of Closure	134
133.	The Revised Method of Closure	135
134.	The Temporary Waste-Weir	136
135.	The Closure of a Large Dam	138
136.	The Preliminary Closure	138
137.	The First Closure	139
138.	The Second Closure	141
139.	The Final Closure	142
140.	Quantities of Work to be done in each Closure	142

XII. *Earthwork Slips and their Repair.*

141.	The Necessity for the Sound Construction of the Dam	144
142.	Faulty Foundations	144
143.	Unequal Settlement	145
144.	Defects in Construction and Design...	146
145.	Changes in Chemical Constitution	146
146.	Infiltration of Water	146
147.	Profile of Slips	146
148.	Slips in Pure Black "Cotton-soil"	147
149.	Slips in Gritty Soil	148
150.	Repair of Slips—Preliminary Operations	148
151.	Repair of Slips—Final Operations	149

XIII. *Pitching.*

152.	The Necessity for Pitching	150
153.	The Laying of Pitching	151
154.	The Extent and Thickness of Pitching	152
155.	Foundation and Top Courses of Pitching	153
156.	Concrete Packing of Pitching	153
157.	Concrete and Brick Pitching...	154
158.	Grass or Reed Revetments	155

CHAPTER III.

The Waste-Weir.

I. *The Weir Proper.*

Para. No.	Subject.	Page No.
159.	Different Forms of Waste-Weir	157
160.	Drowned Weirs and Channels	158
161.	Clear Overfall Weirs	158
162.	Positions of Weirs—Flank and Saddle Weirs	159
163.	Sections of Weirs	161
164.	The Approach Channel	162
165.	Safety Flood Cuts	163
166.	The Depth of the Maximum Flood on the Waste-Weir	164
167.	The Factors Producing the Maximum Rate of Run-off	165
168.	Formulæ for the Rate of Maximum Run-off: Dickens'—Ryves'—Craig's	166
169.	The Limited Utility of Run-off Formulæ—Flood Observations—Cyclones	168
170.	The Objection to Tanks in Series	170
171.	Empirical Allowances for Waste-Weir Runs-off	170
172.	Formulæ for the Discharge of Waste-Weir Channels	172
173.	Calculation of the Discharge of a Waste-Weir Channel	173
174.	The Discharge of a 200-Foot Waste-Weir Channel	174
175.	Formula for the Discharge of Drowned Waste-Weirs	175
176.	Formula for the Discharge of Clear Overfall Waste-Weirs	176

II. *The Tail Channel.*

177.	The Section of the Channel	177
178.	"Retrogression of Levels"	179
179.	The Effect of the Bed Slope upon the Velocity of the Tail Channel Discharge	180
180.	The Outfall of the Tail Channel	181
181.	Design with the Tail Channel Parallel to the Waste-Weir	182

III. *The Flood-absorptive Capacity of Reservoirs.*

182.	The Regulating Power of Lakes	184
183.	The Necessity for Large Absorptive Capacity	184
184.	Method of safely utilizing the Property of Flood-Absorption	185
185.	The Allowance for Flood-Absorption in the Reservoir	186
186.	Examples of Reservoir Flood-Absorption Calculations	188

IV. *The "Stepped Waste-Weir."*

187.	Objections to the Usual Form of Level Waste-Weir	189
188.	General Description of the "Stepped Waste-Weir"	190
189.	Advantages of the "Stepped Waste-Weir"	191
190.	Objections to the "Stepped Waste-Weir"	193
191.	Detailed Description of a "Stepped Waste-Weir"	194
192.	The Under-sluices, Automatic Gates, and Temporary Weir Crest ...	196
193.	Flood Calculation of the "Stepped Waste-Weir"	199
194.	Working of the "Stepped Waste-Weir"	200
195.	Saving Effected by the Adoption of the "Stepped Waste-Weir" ...	201

CHAPTER IV.

THE OUTLET.

I. *General Remarks.*

Para. No.	Subject.	Page No.
196.	The Object of the Outlet	204
197.	The Location of the Outlet	204
198.	The Number of Outlets	206
199.	The Level of the Outlet Sill	206
200.	Subsidiary Uses of the Outlet	207

II. *Different Forms of Outlet.*

201.	Culvert under the Dam	208
202.	General Remarks on Outlet Culverts: Sound Design and Construction Necessary—Watertight Connection with the Trench—Form of the Culvert Arch Ring—The Construction of the Pavement of the Outlet —The Size of the Vent of the Culvert—The Thickness of the Culvert Ring—Staunching Rings	212
203.	Pipe Outlet	215
204.	Siphon over the Dam	215
205.	Headwall in the Centre Line of the Dam	216
206.	The Advantages and Disadvantages of the Headwall in the Centre Line of the Dam	218
207.	Outlet Tower in the Centre Line of the Dam	220
208.	Tunnel round the Dam	221
209.	Headwall across an Open Channel outside the Dam	223
210.	General Remarks: The Outlet Channel—Fore- and Tail-Bays—The Outlet Gauge—Inspection of the Outlet Culvert	223

III. *Regulating Works.*

211.	General Description	225
212.	Outlet Towers	225
213.	Outlet Headwalls	226
214.	The Approach Bank	227
215.	Double Control over Valves	228
216.	Dam Slope Outlet	230

IV. *Valves.*

217.	Ordinary Valves	231
218.	"Pole-and-Plug" Valves	233
219.	Equilibrium Valves—Working of Ordinary Valves	236
220.	Testing Gates—Calculations for Gates	236
221.	Suggested Arrangements for Gates	238
222.	General Remarks: The Outlet as a Flood-Regulator—The Outlet as a Source of Water-Power	239
223.	Lifting Rods and their Adjuncts—Lifting Rods—Guides—Joints—Stops	239
224.	Capstan Heads	241

CHAPTER V.

MISCELLANEOUS.

I. *Water-Supply Schemes.*

Para. No.	Subject.	Page No.
225.	Proximity of Storage to the Town Necessary	243
226.	Amount of Storage, &c., required to Ensure Certainty of Supply ...	243
227.	Sanitation of the Catchment Area—Conservancy of Villages in the Catchment Area—Filtration	244
228.	The Works of a Water-Supply Storage Reservoir	246
229.	Utility of Water-Supply Storage Reservoirs as Famine Relief Works ...	249

II. *Arrangement of Works.*

230.	General Arrangements	249
231.	Works Arrangements	250

III. *Plant Required for the Works.*

232.	General Remarks	252
233.	Tram Plant	252
234.	Rollers	253
235.	Pumps	253
236.	Stone-metal Crushers and Concrete Mixers	254
237.	Tools and Instruments	254

IV. *Maintenance of Works.*

238.	The Dam	254
239.	The Waste-Weir	255
240.	The Outlet	255
241.	The Reservoir	256

V. *The Report, Plans, Estimates and Specifications.*

242.	The Report: General—Preparation of the Project—The Site and the Works—Statistics of Rainfall and Yield—Waste-Weir Floods—Revenue Matters—Financial Return—Arrangement of Report	256
243.	The Plans	259
244.	The Estimates	260
245.	The Specification	261

APPENDICES.

Appendix No.	Subject.	Reference.		Page No.
		Chapter No.	Paragraph No.	
1.	Comparative Statement of Dimensions and Cost of Storage Works with Earthen Dams in the Bombay Presidency (Deccán) ...	I.	28	262
2.	Table of Total Monsoon Rainfall and Estimated Run-off and Yield per Square Mile from Catchment Areas	I.	16	263
3.	Calculation of the Amount of Storage required for the Irrigation of a Certain Area and to Supplement the discharge of a Natural Stream	I.	26	264
4.	Tank Expenditure Estimate	I.	27	266
5.	Estimate of the Duration of a Water-Supply Storage	I.	27	268
6.	Statistics of certain Irrigation Works in India	I.	29—31	270
7.	Estimates of Revenue Results from Tank Storage	I.	29, 32	274
8.	Table of Waste-Weir Runs-off	III.	171	277
9.	Table of Values of Bazin's Co-efficients for Earthen Channels	III.	172	278
10.	Table of the Discharge of a Waste-Weir Channel having a Bed Width of 200 feet and a Bed Slope of 1 in 100	III.	174	279
11.	Table of the Discharge from each Foot of Length of a clear Overfall Weir in Cubic Feet per second	III.	176	280
12.	Máládevi Tank Project. Temporary Waste-Weir Flood Calculations—First Closure ...	II.	137	282
13.	Máládevi Tank Project. Temporary Waste-Weir Flood Calculations—Second Closure ...	II.	138	284
14.	Máládevi Tank Project. Permanent Waste-Weir Flood Calculations	III.	193	286
15.	Recapitulation of Máládevi Tank Project ...	V.	244	288
16.	Abstract Estimate of Máládevi Tank Project	V.	244	289
17.	Máládevi Tank Project. Table of Reservoir Contents at each Contour	I.	21	294
18.	Specifications for the Puddle Trench, Dam Embankment and Pitching	V.	245	297
19.	Tables of the Cross-Sectional Areas of Puddle Trenches	II.	92	320
20.	Tables of the Cross Sectional Areas of Dam Embankments	II.	73	322
21.	Tables of the Cross Sectional Lengths of Pitching	II.	152	327
22.	Notes on the Arrangements for and Management of Large Works	II.	126 (*c*)	330
23.	Table of Task-work in different Soils	—	—	364
24.	Tables for Ascertaining the Cost of Carriage by Coolies and by Carts	—	—	369
25.	Useful Memoranda	—	—	372
26.	Statistical Information required for a Project	V.	242 (*a*)	375

LIST OF ILLUSTRATIONS.

Plate No.

1. DIAGRAMS.

1. Diagram showing Approximate Amounts of Percentage of Run-off to Rainfall, Depth of Run-off due to Rainfall, and Yield of Run-off from Good, Average, and Bad Catchments.
2. Diagram of Rate of Waste-Weir Runs-off in inches per hour.

2. MÁLÁDEVI TANK PROJECT.

3. General Plan, showing Catchment Area and Reservoir.
4. Dam Embankment, General Plan and Longitudinal Section.
5. „ „ Cross Sections.
6. Waste-Weir, General Drawings.
7. „ Enlarged General Drawings.
8. „ Detailed Drawings.
9. Outlet, General Drawings.
10. „ Detailed Drawings.

3. MISCELLANEOUS DRAWINGS.

11. Dhupdál Storage Reservoir : Design for 7′ 0″ × 4′ 6′ Sluice Gate.
12. Waghád Tank Outlet Sluice : Details of Ironwork. Bateman's Capstan Head.
13. Hubli Waterworks—Outlet : General Drawings.
14. „ „ „ Detailed Drawings of Ironwork.

LIST OF FIGURES IN THE TEXT.

CHAPTER II.

Figure No.	Subject.	Page No.
1.	Core Wall of New Croton Dam	55
2.	Plan of Berm	74
3, 4.	Sections of Toe Walls	75
5.	Ordinary Type Sections of Dam 60 feet high	78
6, 7, 8.	Puddle Trench Sections	95
9, 10.	Proposed Bohio Dam, Panama Canal, longitudinal and cross sections	100
11.	Section of Dam proposed by Mr. Kreuter	108
12.	Cross-sectional Junction of Embankment	124
13.	Longitudinal Junction of Embankment	125
14.	Setting-out Template	128
15.	Usual Closure of a Dam Embankment	134
16.	Proposed Closure of a Dam Embankment	135
17.	Cross Sections of a Slip of a Dam Embankment	147
18, 19.	Foundation and Top Courses of Pitching	153

CHAPTER III.

20.	Flank of Waste-Weir, Plan and Section of	160
21–24.	Sections of Waste-Weir Walls	161
25.	Flood Section of a Waste-Weir Channel	171
26.	Flood Section of a Drowned Waste-Weir	175
27.	Flood Section of a Clear Overfall Waste-Weir	176
28.	Plan of a Waste-Weir Channel	177
29.	Section of Waste-Weir Water Cushions in Series	179
30.	Plan of Waste-Weir with parallel Tail Channel...	183
31.	Design of a "Stepped Water-Weir" for a Flank Weir	190
32.	Design of a "Stepped Waste-Weir" for an uncertain Catchment ...	191
33.	Cross Section of an Automatic Gate and Opening	197

CHAPTER IV.

34.	Outlet, Headwall and Approach Bank	209
35.	Section of Junction of Outlet Tower and Culvert	211
36.	Longitudinal Section of Outlet Siphon	216
37.	Plan of "Dry-well"...	220
38.	Plan of Outlet Tunnel	222
39.	Plan of Outlet Headwall across an Open Channel	223

CHAPTER IV.—*continued.*

Figure No.	Subject.	Page No.
40, 41.	Plans of Outlet Approach Banks	227
42.	Headwall in Centre Line of Dam, Plan and Section	229
43.	Dam-Slope Outlet, Plan and Section	231
44.	"Pole-and-Plug" Outlet Headwall	232
45.	Iron "Pole-and-Plug" Valve	233
46.	"Pole-and-Plug" Outlet Tower	234
47.	"Pole-and-Plug" Outlet Tower	234
48.	"Pole-and-Plug" Dam-Slope Outlet	235
49.	Equilibrium Valve, Diagram of	236
50.	Test of a Sluice Gate, Diagram of Forces	236

CHAPTER V.

51.	Outlet Pipe-hanger	247
52.	Plan of Inlet Filter	248
53.	Plan of Setting-out Borrow-pits	251

GLOSSARY OF ABBREVIATIONS AND INDIAN WORDS.

Cusecs	...	...	= cubic feet per second.
F. S.	...	...	= full-supply.
F. S. L.	...	...	= full-supply level.
H. F. L.	...	...	= high-flood level.
M. S. L.	...	...	= mean sea level.
R. L.	...	...	= reduced level.
Mill. cft.	...	...	= million cubic feet.
Rft.	...	...	= running feet.
Sft.	...	...	= square feet.

Anna	...	...	= $\frac{1}{16}$th rupee = 1*d.*
Bábhul	...	...	= a common hard and tough wood (Acacia Arabica).
Bhistie	...	...	= a water-carrier.
Coolie	...	...	= a works labourer.
Ghát	...	...	= main mountain range.
Kankar	...	...	= nodular hydraulic limestone.
Karal	...	...	= a marly soil charged with salts.
Kárkun	...	...	= a works clerk or timekeeper.
Kharif	...	...	= the season from February 15th to October 14th.
Maistry	...	...	= a works foreman.
Mán	...	...	= a hard clay soil.
Monsoon	...	...	= the rainy season.
Muccadum		...	= a works gang-man.
Murum	...	...	= disintegrated trap rock.
Nulla	...	...	= a water-course.
One lakh	...	...	= 100,000.
Peon	...	...	= an office messenger.
Pie	...	...	= $\frac{1}{12}$th anna = $\frac{1}{192}$th penny.
Rabi	...	...	= the season from October 15th to February 14th.
Rupee	...	...	= 1*s.* 4*d.*
Súp	...	...	= a basket scoop for bailing water.
Tamarisk	...	...	= a jointed evergreen reedy plant growing on riverain lands
Tank	...	...	= a storage reservoir.

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INDIAN STORAGE RESERVOIRS WITH EARTHEN DAMS.

CHAPTER I.

THE RESERVOIR.

I. Selection of the Site.

1. Comparison of Near and Distant Reservoirs.—In respect to their position, reservoirs may be classified as being either near to, (say within 10 miles of), or far from, the area to be irrigated by them. Compared with distant reservoirs, near ones have the following advantages:—It is easier to arrange for regulating the supplies which are required from them, and generally to manage them, and their supervision can be effected at a cheap rate. They have, however, compared with distant reservoirs, the following disadvantages:—The country in which they are situated will be generally of a more open character and will have less pronounced ridges, thus involving greater expense in construction; the land to be submerged will be usually of a higher quality and thus more costly to acquire; and, finally, the rainfall is most likely to be less in amount and more uncertain in character, while the proportion which runs off from the catchment area, owing to its more cultivated character and gentler slopes, as a rule will be less. This last is the greatest disadvantage, as

an assured and ample replenishment is a matter of the first importance.

2. Loss in Transit down Natural Stream Beds.—There is another advantage which near reservoirs possess, and that is that there is very little loss of water in transit from them to the headworks of the canals which they supply, and, when these take off immediately from them, there is, of course, no such loss. The loss in transit is due partly to evaporation from the exposed water surface and partly to absorption by the bed of the channel down which the supply flows. What the loss may amount to it is impossible to estimate exactly in the absence of careful experiments, and it is difficult to arrange for these before the works are constructed, as usually water will not be available during the hot dry season in which afterwards it will have to be passed down. The best thing to be done is to gauge the natural stream at different points as late in the hot weather as possible, and thus to determine the losses between these selected places.

If the supply is to be let down the natural channel of the feeding river, everything will depend upon its bed—if that is sandy, the loss may be very great; whereas, if it is rocky, it may be small, as the strata may be naturally charged with subsoil water, and thus not greatly absorbent. A tortuous channel will increase the loss both by its increased length, leading to increased absorption, and by the increased surface it will offer to evaporation; it will be, however, but rarely that the natural course can be improved at an economical cost, as the improvements made in one year are liable to be destroyed in the subsequent monsoon.

3. Loss in Transit down Canals.—If the supply is passed down to the irrigated area by a channel, the loss in transit will be generally less than it would be if it were let down the river bed, but, in this case, the cost of the

connecting works would have to be taken into account. Certain experiments of the loss of water in transit which have been made on canals are detailed in paragraph 24 below. The loss on canals diminishes greatly after they have been constructed for some time, as their wetted section consolidates and becomes puddled by silt in the water.

4. Selection of the Site of the Dam.—In choosing a site for a dam it should be remembered that the longitudinal section generally determines the cost of the scheme; the sites available for waste-weirs, its feasibility; and the nature of the tank basin, whether open or restricted, the relative cost of storage.

The best site for a dam is usually one which has ridges running down from high land on both sides to the stream to be impounded; these ridges will very greatly reduce the cost, as the section of a dam varies roughly as the square of its height; low, long dams are thus frequently cheaper than short, high ones, and are also much safer.

A ridge, moreover, offers facilities for the proper drainage of the dam; but it should not be very narrow, for, if it is, this may render it liable to leak; may not allow space for the future raising of the embankment, should that ever become necessary; and may also lead to greater cost and difficulty in construction.

It should also be seen that the longitudinal section provides proper sites for the outlet, temporary (*vide* Chapter II., paragraph 134) and permanent waste-weirs, and for "breaching sections" of the dam (*vide* Chapter II., paragraph 75), at which, in case of its being breached, the resulting flood will do the least amount of damage. Generally the best site is where the gorge part of the dam is separated from the flank embankment by a hillock rising above the level of the top of the dam. The two portions of the dam can be thus completed independently of each other. This formation is peculiarly


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suited for composite dams, as the gorge portion can be constructed in masonry, while the flank can be made in embankment.

The abundance of all materials required for construction—such as the proper class of soil for the embankment, water, sand, lime, wood for fuel, and stone for masonry and pitching—should be also taken into account. It is generally impossible for economical reasons, to construct an ordinary earthen dam if sufficient good soil does not exist within half a mile of its site.

5. Selection of the Storage Basin.—After a site has been found which satisfies purely replenishment and constructional requirements, the next thing to ascertain is its storage capability. The cross-sectional slopes of the country can be easily seen by inspection, but the longitudinal slope, which is far more important, may not be so readily ascertainable, as, not only is it much longer, but also it may be concealed by the windings of the valley. It is upon this latter slope that the extent of storage chiefly depends, and the most capacious reservoirs having a moderate cost of storage are generally long and narrow ones. Some excellent ones are shown in plan in Schuyler's book on reservoirs—the reservoir basins in the western part of the United States appear to be conspicuously good ones.

The longitudinal slope of the basin will be generally similar to that of the river, and any variation of the two can be easily seen by noting the heights of the river banks. The fall of the river in tank limits can be determined roughly by an inspection of its pools and rapids; if, however, the river course is tortuous, this fact must be taken into account, as it will then generally have a flatter slope than that of the country. As a rule, the tank basin will be found to be flat at the head of rapids in the river, especially if it is of ancient lacustrine origin. The longitudinal section

of the valley can be often ascertained by an inspection of its topographical map : it will be generally flat if the river is shown to have windings of large curvature ; if, on the other hand, the river is straight, or has numerous sharp bends, these are indications that its fall is rapid. One of the most favourable sites for a dam is below the junction of two streams, as then storage will be obtained up both their valleys ; such a place is frequently an indication of good foundations (as hard substrata may have deflected the tributary into the main river), and also that the ridge lines on the opposite sides of the valley approach each other.

The longitudinal slope of the river within the tank basin varies in Bombay examples from 2·5 feet to 61·00 feet a mile. In respect to storage capacity, a slope less than 10 feet a mile may be considered a good one ; one from 10 feet to 20 feet a mile, a fair one ; and one exceeding 20 feet a mile, a poor one. In all cases the effect of the longitudinal slope may be modified by the nature of the side slopes and by the existence of tributary basins.

II. The Catchment Area.

6. Classification of Tank Catchments.—Tank catchments may be classified into those which are :—

(*a*) Wholly natural ;

(*b*) Partly natural, partly artificial ;

(*c*) Wholly artificial.

A wholly natural catchment is one in which the dam intercepts all the drainage which replenishes the tank without the aid of subsidiary works. It is, as a rule, the cheapest variety, both in capital cost and annual maintenance.

A partly natural and partly artificial catchment is one in which the natural catchment is supplemented by the

entire, or partial, diversion of the drainage from neighbouring catchments physically separated from it. It will be generally dearer in capital cost than (*a*), unless, which is not often likely, the cost of leading the drainage from the subsidiary catchments into the main one is less than would be entailed by damming the latter either lower down or at a less favourable site. The cost of maintenance will be almost always higher than that of (*a*). Such aided catchments are usually not desirable, except in the case of water-supply schemes, for which the sites for tanks are limited to those in proximity to the population to be served, and for which a certain replenishment in all years is a *sine quâ non*.

A wholly artificial catchment is one where the storage is effected in a basin without any appreciable extent of drainage area of its own, and which has, therefore, entirely to be fed from a neighbouring area. Should this be arranged for by a channel from a large stream, it will be the safest system to work, as it will be most under control, and, if that stream is one with an ample and unfailing supply, this form will give the most certain replenishment. It will be, however, as a rule, the most costly class to construct and to maintain.

7. Yield from the Catchment.—The yield from a catchment depends upon its size; the nature of the surface, whether hilly or flat, barren or cultivated, treeless or wooded, impermeable or permeable; the amount and intensity of the heavy falls of rain, which produce practically all the run-off; and the general conformation of the area and the position it occupies with respect to the direction of the tank-filling storms. In regard to the last it may be noted that tanks have been known to have bad replenishments because their catchments are to the leeside of ranges of hills, on which side there is but little precipitation. In Appendix 2 are given approximate

calculations of the amount of the yield of typical catchments (see also paragraph 16).

The proper replenishment of a tank is the most important consideration in connection with it, for, with a deficient storage, the utility of a reservoir is diminished. The catchment should, therefore, be of sufficient extent to give the required full storage in an ordinary bad year. It is not necessary that it should be large enough to fill the tank during an exceptionally bad famine year, as then even the largest areas, if situated in a famine-stricken country, will probably fail to produce a sufficient replenishment. It may be that the tank is not required in years of ordinary rainfall, and may fail in years of drought. Such conditions may be met best by placing the storage in an area where the replenishment in all years is certain, *e.g.*, near the western ghâts in Bombay, and all such areas should be assuredly utilised to the greatest extent possible.

There are, however, cases where a moderate amount of storage may be beneficial to a large area of irrigation by providing supply to tide over breaks in the monsoon, and such storage may be obtained in the plains in years of light rainfall, provided the catchment area is of fair extent. On the other hand, as irrigation schemes should be fairly remunerative, excessively large catchment areas should be avoided, as these entail large waste-weirs and involve very rapid silting up of the tank basin.

8. Partly Natural and Partly Artificial Catchments. —Equivalent Catchment Area.—Tanks of class (*b*) usually depend upon the assistance afforded by intercepting the drainage from subsidiary areas by means of feed channels. It is, however, economically impossible to construct feed channels of any length of a section sufficiently large to carry the whole run-off due to the heavy falls of rain, which are the ones chiefly productive of replenishment. Moreover, with

a very large section there is the chance that a considerable portion of the run-off from light showers will be lost in transit by absorption in the bed of the channel. It will be, therefore, generally advisable to design the feed channel of a size sufficient to carry one-eighth of the estimated run-off produced by the downpours of greatest intensity and to calculate on its diverting not more than one-fourth of the whole run-off of the year. By dividing the area of the subsidiary catchment by four and by adding this to the natural catchment is obtained what may be called the "equivalent catchment" of the tank. Unless the equivalent catchment of the subsidiary drainage area bears a fair proportion to the natural catchment of the main drainage area, it will not, as a rule, be advisable thus to increase the replenishment of the tank. It must be always remembered that these minor areas may have a yield not proportional to that of the main one. They may be passed over by the heavy tank-filling rain, or may have flatter slopes, or a more absorbent surface than it has.

Feeder channels, from their nature, are peculiarly liable to damage in heavy storms, and may breach during their continuance and thus fail to intercept the run-off. They are further likely to cost an amount for maintenance largely disproportionate to their tank-filling value.

In some old native examples channels have been made to serve a double purpose, *i.e.*, as feeder channels to increase the replenishment of the tank during the monsoon and, thereafter, as irrigation channels from it. This is not a desirable practice to follow, as the bed levels will be against it; either the channel will have to be headed up to a considerable extent at the tail in order to admit the water into the tank, or, the slope of the channel will have to be so flat that flow in either direction can be produced by a moderate amount of heading up at one end or the other. In either case it

is probable that a considerable amount of silting will be caused, and an otherwise unnecessarily large section and heavy embankments will be required.

9. Subsidiary Reservoirs.—There is another device by which the rainfall from subsidiary catchments may be diverted more effectually into the main reservoir, and that is by forming minor reservoirs at each crossing of the minor streams, thus impounding them. It will suffice to make these smaller tanks of a capacity only large enough to contain the run-off due to the ordinary maximum daily rainfall, as, during the intervals between falls of rain, their storage can be run down the connecting channel to the main tank, and they will thus regain capacity for storing the run-off due to the next fall of rain. This system of minor tanks is decidedly better than that of the simple feeder channel, as by it practically all the run-off due to their catchments in ordinary years may be passed into the main tank. Moreover, should the connecting channel be damaged at any time, the supply may be shut off from passing down it until repairs have been effected. On the other hand, the capital cost involved by it will be generally greater than that of the simple feeder channel, even although the size of the connecting channel is considerably reduced by being designed to carry only the average supply from the subsidiary drainage area.

Occasionally the subsidiary reservoir may have to be formed downstream of the main one. In this case the working of the system will have to be reversed. The subsidiary reservoir should then have a capacity sufficient to intercept the run-off due to its catchment during a few days of successive rainfall, and to supply the irrigated area during the subsequent break in the monsoon. In this latter time it will be run dry, and thus will be ready to store the next series of replenishments, and, if its capacity

is not sufficient to tide over a break, it can be supplemented by supply from the main tank. The main tank would be reserved for this purpose, and chiefly for the requirements of irrigation after the close of the monsoon, as evaporation from it will be relatively less than that from the shallower, subsidiary reservoir.

10. Wholly Artificial Catchments.—Tanks of class (*c*) will be supplied generally from large rivers, on which weirs will be constructed, and the required discharge will be led from them by means of connecting channels to the reservoirs. Such systems will usually be the most expensive of all to construct, but in certain cases they may be the only ones practicable; in others they may be economical, as the capacity of the reservoirs can be reduced so as to be only large enough to tide over the period when the natural discharge of the parent streams will not suffice for the requirements of irrigation—the shorter this period, the less the capacity required. They are of great utility in insuring the irrigation of perennial crops, which would be otherwise impossible from the natural streams, and will be frequently of great use in years of deficient rainfall, as the streams which supply them will usually have even then a total run-off sufficient for the area under command if a large proportion of it is impounded during the monsoon.

11. Measurement of Catchment Area.—To determine the catchment area, it will generally suffice to mark the watershed on the topographical maps, and to measure the enclosed drainage area either by ruling squares over it or by means of a planimeter. Where, however, the natural features are undefined on the maps, the watershed line should be marked on them by actual inspection and survey of the ground.

The most exact way of calculating the run-off efficiency of the catchment is to divide it into areas on which the

rainfall will be of similar amount, or the slopes and general character of the surface are the same. The average run-off from each constituent area can be then determined, and the total run-off will be equal to their sum. It is, however, not usual to work out this in detail, an average rate of run-off being generally estimated for the whole catchment area.

III. Rainfall and Run-off.

12. Natural Disposal of the Rainfall.—When rain falls on a catchment, a part of it is absorbed by the soil and is lost; another part penetrates more deeply and produces springs, but in India these generally give an insignificant amount of discharge; and another part remains on the surface until it is dissipated by evaporation. Finally, another part at once runs off the surface to the drainage lines; it is this which produces the monsoon flow of streams, and constitutes the large bulk of the supply available for replenishing tanks. The proportions in which the total rainfall will be thus disposed of will depend upon the nature of the surface (as already described in paragraph 7), the intensity of the showers and the intervals at which they occur.

13. Variations in Rainfall and determination of the Proportion of Run-off.—In England, where the total annual rainfall does not vary greatly, the following laws have been put forward. "Glaisher's law" is that the average fall of three consecutive years yielding the least fall may be taken as the average minimum fall. "Hawksley's law" is—from the average fall of twenty years deduct one-sixth for the average of the three years of minimum fall. These two laws should agree in any selected case.

In India such general rules have not been devised, and, if devised, would probably not hold good owing to the

extreme variations that exist—on the one hand between famine years, when there may be an almost total absence of rain, and, on the other, years of excessive fall, when the precipitation may be many times that during an average year.

In years of deficient fall, to mitigate the effects of which storage reservoirs are chiefly required, not only is the total precipitation small, but the proportion of it which runs off, compared to that of average years, rapidly diminishes. The behaviour of each catchment in this respect requires individual study, as each has its local peculiarities, but much might be learnt from that of similarly situated catchments generally resembling the one under consideration. It is therefore highly important to maintain observations relating to this subject on all existing works, and, for as long as possible, in connection with proposed works.

In the former case it is easy to measure the total run-off by the replenishment received until the time when the reservoir fills, and this is done on works in the Bombay Presidency. It is, however, equally, if not more, important to continue the observations throughout the whole year, but this has not been usually done there. In order to ascertain the run-off during the whole year it will be necessary to obtain the total discharge passed off by the waste-weir, either by frequent observations of the depth flowing over it, or by maintaining a continuous diagram of these depths by means of a self-recording instrument, such as the "automatic water-level recorder," made by the Glenfield Co., Kilmarnock. From such a diagram another diagram of discharges could be prepared by plotting the rate of discharge corresponding to each depth on the weir crest, and the total discharge could be ascertained by measuring the area included by this second diagram. Or, a recorder designed to register the amount of the discharge automatically (also made by the same firm) might be used.

In the latter case the total run-off of the year is ascertained by gauging the stream once or twice daily during ordinary flow, and at more frequent intervals during floods, when the level changes rapidly. It is, however, difficult to arrange for such observations at night and during floods of great intensity.

14. Necessity for maintaining several Rain-gauges.—To compare the rainfall with the run-off it is not sufficient to maintain a single rain-gauge. At the Kás tank, in the Satára district, Bombay, although the catchment is very small (1·97 square miles), the rain-gauge, which is at the site of the dam, is not representative of it owing to the great variation of the rainfall, which is due to the fact that a part of the watershed is situated on the crest of the ghâts. The result is that the total annual run-off sometimes exceeds the amount which would be produced were the gauged rainfall uniform over the catchment and wholly to flow off it. Generally speaking, in hilly tracts the rainfall at the dam site is always considerably less than the average fall on the catchment. It is therefore necessary in order to ascertain this average fall, to have numerous rain-gauges distributed over the catchment.

15. Examples of the Proportion of Run-off to Rainfall.—A few instances are given below of the percentage of run-off to rainfall.

Observations[1] recorded at Nágpur and at Jubbulpore, in the Central Provinces, showed that in ordinary years the total run-off from a rocky catchment of 5 or 6 square miles was 40 per cent. of the total rainfall. At the Oopahalli tank, in Mysore, the total run-off from 1878 to 1883 varied from 10 to 19 per cent. only of the total rainfall; in the Hulsar tank, at Bangalore, during three years the percentage was 14.

[1] Buckley's "Irrigation Works in India and Egypt," pp. 63 and 64.

In the Bombay Administration Report of the Public Works Department for 1890—91 is a statement showing the behaviour of the catchments of 19 tanks in the Deccán during the monsoon rainfall, which varied generally from 20 to 30 inches, but fell in some cases to as little as 4 or 5 inches. The 114 observations showed that there were:—

26	cases where the flow-off was		less than 10	}	per cent.
44	,,	,,	,, between 10 & 20		of the
25	,,	,,	,, ,, 20 & 30		total
19	,,	,,	,, above 30		rainfall.

In Bombay it is generally assumed that the run-off will be only 25 per cent. of the monsoon rainfall, when this usually varies from 20 to 25 inches.

The following record of observations is taken from the Bombay Irrigation Revenue Reports:—[1]

Year.	Mhasvád Tank. Catchment Area, 484 sq. miles.		Ashti Tank. Catchment Area 92 sq. miles.		Ekruk Tank. Catchment Area, 159 sq. miles.	
	Monsoon Rainfall. Inches.	Percentage of Run-off.	Monsoon Rainfall. Inches.	Percentage of Run-off.	Monsoon Rainfall. Inches.	Percentage of Run-off.
1884—85 ...	13·40	9·5	Abnormally small	1·9	20·39	17·8
1885—86 ...	20·01	28·0	28·91	24·5	27·93	21·5
1886—87 ...	28·88	15·5	29·21	16·5	33·05	22·6
1887—88 ...	27·76	12·1	18·16	18·3	35·88	33·8
1888—89 ...	21·00	10·5	13·89	11·7	24·75	19·8
Average ...	22·21	14·34	22·54	18·69	28·40	24·04

These tanks are situated in the "Famine Zone." It will be noticed how small is the run-off percentage, and that in a year of abnormally small rainfall it practically vanishes; also,

[1] Buckley's "Irrigation Works in India and Egypt," p. 64.

how, (owing to their physical peculiarities), the average percentage run-off differs considerably for the three catchments although the average rainfall on them does not vary greatly.

The following record is taken from the report of the Rushikulya project in Madras :— [1]

Year	Mahanadi. Catchment, 900 sq. miles.		Rushikulya. Catchment, 850 sq. miles.	
	Rainfall of the Year. Inches.	Percentage of Run-off.	Rainfall of the Year. Inches.	Percentage of Run-off.
1868	44·1	5·0	44·1	6·3
1869	59·6	36·1	59·6	31·0
1870	55·5	51·3	52·7	74·2
1871	43·7	18·0	42·0	8·4
Average	50·73	29·60	49·6	32·2

The above shows that the percentage of run-off generally decreases with the total annual rainfall, but that it does not always do so. This point is noticed in paragraph 17 below.

The following table [2] shows the effect of the state of saturation of the ground on the percentage of run-off. The observations were made on a catchment of about 100 square miles near Calcutta :—

Month.	Percentage of Run-off to Rainfall during the Month.		Remarks.
	In Years of Ordinary Rainfall.	In Years of Heavy Rainfall.	
June	5·0	10·0	At end of hot weather.
July	10·0	20·0	
August ...	25·0	50·0	Monsoon well established.
September ...	40·0	50·0	
October ...	40·0	50·0	End of Monsoon.

[1] Buckley's "Irrigation Works in India and Egypt," p. 64.

[2] *Ibid.*, p. 65.

16. Tabular Statement of Rainfall and Run-off.—Careful experiments were made during the monsoon of 1872 by Mr. (now Sir Alexander) Binnie on the Ambájhari reservoir catchment near Nágpur, in the Central Provinces, and a table of the percentages of run-off to rainfall was published[1] by him. In Appendix 2 is a similar table in which the percentages are given for a regularly ascending scale of rainfall, and under three conditions, viz., of a bad, of an average, and of a good catchment. The results are plotted in the diagram, Plate I.

This classification of catchments relates to their capabilities of producing runs-off, and the figures for a good catchment approximate to those given for Ambájhari. The table is based on general ideas, and it is hoped it will prove useful. It would be of great utility were similar tables constructed by engineers for different actual catchments. The general result to be noticed from this table is that in years of deficient rainfall the percentage of run-off rapidly diminishes, so that, on account both of the smallness of the fall and of the percentage of run-off, the replenishment of a tank in such years is very small when compared with that in a year of good rainfall, or even in one of average fall.

17. The Effect of the Intensity of the Rainfall and the Condition of the Ground on the Run-off.—The chief defect of such a table is that it is based on the annual fall, and not on the intensity of the individual showers of rain which actually produced the run-off, and the condition of the catchment when those showers fell. Thus, considering the same catchment, two years with identically the same total rainfall may produce replenishments of greatly differing amount. It is evident that if the total rainfall is fairly distributed over sixty days it will produce much less

[1] "Minutes of Proceedings, Inst. C.E.," Vol. xxxix.

run-off than if it fell during thirty days, and, still less, than if it occurred during only fifteen days. The proper way, therefore, is to take into account the intensity, or rate of fall, of the rainfall. It would be best to consider the hourly intensity, but, as this is seldom recorded, the daily intensity may, in default, be taken.

The intensity of the rainfall is not, however, the sole factor which determines the rate of run-off. Cases have been known where even heavy falls of rain at the beginning of the monsoon, or after long breaks in it, when the surface was dry and absorbent, have not produced any run-off whatever, while falls of the same or of less amount, when the soil was saturated, have caused floods. It is evident, therefore, that the degree of saturation of the surface must be also taken into account when estimating the proportion which the run-off will bear to the rainfall.

18. Tabular Statement of Daily Runs-off.—As far as is known, careful observations of this latter effect have not

TABLE OF DAILY RUNS-OFF.

1	2		3		4	
Daily Rainfall in inches.	Run-off Percentage and Yield when the original state of the ground was					
	Dry.		Damp.		Wet.	
	Per-centage.	Yield in inches.	Per-centage.	Yield in inches.	Per-centage.	Yield in inches.
0·25	—	—	—	—	12	0·03
0·50	—	—	10	0·05	14	0·07
0·75	—	—	12	0·09	17	0·13
1·00	5	0·05	14	0·14	20	0·20
1.25	6	0·08	16	0·20	23	0·29
1·50	7	0·11	19	0·29	26	0·39
1·75	8	0·14	22	0·39	30	0·53
2·00	10	0·20	25	0·50	34	0·68
2·50	15	0·38	32	0·80	43	1·08
3·00	20	0·60	40	1·20	55	1·65
4·00 and over	30—40	1·20—	50—60	2·00—	70—80	2·80—

been recorded. The percentages of run-off will of course vary with the other conditions of the catchment, but the table on p. 17 is given as a rough approximation of what may be expected from an ordinary drainage area. The percentages will increase with a good, and diminish with a bad, one.

In this table, the percentages of run-off from wet surfaces, compared with those from dry surfaces, are proportionately greater for light falls than they are for heavy falls. In the latter case, the surfaces originally dry will have become saturated before the falls have ceased, and the conditions producing run-off from both classes of surface will thus be more alike. Compared with each other, the percentages of run-off from each state of the ground increase directly with the intensity of the rainfall.

19. Estimation of the Run-off from the Daily Rainfall.—The proper way to estimate the run-off, in the absence of observations of it, will be to proceed on the lines of the above table, and, if a few discharge observations have been taken, to construct the table with reference to them. If the rainfall for a series of years is thus dealt with, a very fair idea of the sufficiency, or not, of a catchment can be arrived at. If it is ascertained that during years of average rainfall the tank will probably fill, and that in years of scanty rainfall, not amounting to absolute deficiency, there will be a fair replenishment, the catchment may be considered a sufficiently large one. As such a determination will be based on estimates of the percentages of run-off, care should be taken not to over-estimate them.

Another way of estimating the sufficiency of a catchment is to take the average annual amount of the heavy falls of rain, each exceeding one inch, during a series of years, excluding years both of excessive and of greatly deficient fall, and to apply a general percentage for run-off to this. If the result shows that the catchment is likely to produce the

required replenishment, it may be accepted as sufficiently large. This method is less laborious than the former, but, in proportion to its diminished detail, is less likely to be accurate.

Such estimates are but approximations, and should be resorted to only in the absence of actual discharge observations. It is a matter of the first importance to have such observations for as many years as possible, and, therefore, no large scheme should be proceeded with until a record of at least ten years has been obtained. Of course, where they exist, the results of observations for other similar and similarly situated catchments may be utilised to diminish the period of examination. Even when this can be done, it is advisable for large schemes to institute comparative, concurrent observations of the run-off of the catchments to be dealt with, with those for which observations have already been recorded, so as to determine what factor should be applied to the previous results of the latter to enable them to be utilised for the former.

IV. The Storage Capacity of Tanks.

20. Proportion of the Storage Capacity to the Yield from the Catchment.—It is necessary to determine the storage capacity of the tank by reference chiefly to the run-off from the catchment which can be impounded, as, in the large majority of cases, the area which can be commanded will be in excess of that which can be irrigated by the scheme. Where this is not the case and the area of irrigable land is limited, its extent has also to be taken into account in the manner described in paragraph 26 below.

It will be best to limit the storage capacity to that which can be replenished in an ordinary year, as it is desirable in irrigation from tanks that the irrigated area should not greatly fluctuate. If it is made much greater than this, it will be only in good years that the tank will

fill, and, in all other years, there will be a certain amount of unproductive capital expenditure due to the increased size of the works beyond the requirements of the then storage. A small increase of storage in excess of the average amount of replenishment, *i.e.*, one not exceeding 10 per cent., is, however, desirable to compensate for the reduction of capacity that will occur by the silting up of the tank basin. It is not necessary greatly to anticipate the future in this respect, as the original storage, when much reduced by this cause, may be restored by raising the full-supply level and the dam. It is thus not desirable at first to incur much increase of capital expenditure on this account as it will be infructuous for many years.

If the storage capacity is originally made much less than the average replenishment, there will be a diminished rate of return on the capital, as the cost of storage per mill. cft. usually decreases with its increase in quantity. There will be also an increase in the proportion of silt to storage.

If monsoon irrigation is to be practised under the tank, the quantity of water required for it should be taken into account. The catchment will in this case produce a certain amount of replenishment, of which part will be expended during the monsoon, and the size of the tank should therefore be limited so as to store only the remainder.

21. Calculation of Storage Capacity.—The calculation of the storage contents of a reservoir is made by summing up the contents between each of its contours. The formula to be used for this is:—

$$Q = \frac{H}{3}(A + a + \sqrt{A \times a}),$$

where Q is the storage in cubic feet;

A is the area of a contour in square feet;

a is that of the adjacent contour in square feet; and

H is the vertical distance between them in feet.

Although it is unnecessary that contours should be surveyed at small vertical distances apart, the calculation of the storage will be more exact if such contours are taken into account. The general practice is to survey contours at vertical differences of 10 feet and to interpolate others at 1 foot intervals. To determine the areas of these interpolated contours, it is usual to take the square roots of the surveyed contours and to consider that the square roots of the interpolated ones vary in exact proportion to their vertical distances apart from the others. A calculation of the capacity of a tank is given in Appendix 17, and this shows the method followed in ascertaining the areas of the interpolated contours and the storage contents between them.

In determining the total storage capacity of a reservoir, it is not usual to add the further capacity due to the excavation of the borrow pits which furnish material for the dam. The contents of these will be relatively very small to the amount of natural storage, and, in respect to the capacity available for irrigation, will be still smaller, as a large proportion of them will be below outlet sill level.

From the gross capacity of the reservoir has to be deducted the storage below outlet sill level, as this cannot be utilised for irrigation except by pumping, which, in practically all cases, is too expensive a measure to which to resort. The balance storage—the quantity above outlet sill level—is known as the "available capacity," and this is all that can be taken into account when determining the irrigating capability of the reservoir. As it is not likely in most cases that the reservoir will fall below outlet sill level before the commencement of the monsoon, it will be generally quite safe to take only its "available capacity" into account when considering the total amount of probable replenishment.

22. Loss by Evaporation and Absorption.—In order to ascertain the irrigating capability of a reservoir, it is necessary to deduct from its "available capacity" an allowance for evaporation and absorption. The total loss on these accounts can be measured by noting the total fall of the reservoir surface, and by deducting from the loss of storage thus indicated the actual, measured amount utilised for irrigation. The estimation of the amount of loss due to each of these causes is not so easy to determine. The loss due to evaporation in an experimental cistern can, of course, be easily measured, but this will be due solely to the effect of the temperature of the air on the surface of the water and on the sides of the cistern. In the case of a large body of water, such as a reservoir, the amount of evaporation greatly depends also upon the drying effect of wind, (producing waves and spray), which will not materially affect the contents of a small cistern, and cannot therefore be ascertained by its use. For this reason it is usual to take together the amounts of the losses due to both evaporation and absorption. In respect to evaporation alone, the loss will be in direct ratio to the area of the water surface and in inverse ratio to the depth, as the temperature of shallow water is always higher than that of deep water.

The loss due to absorption alone depends upon the nature of the bed of the tank basin and the depth of the storage. It will increase with the porosity of the underlying strata, and with the pressure due to the depth of water. In ordinary cases, and more especially after some years when the bed of the tank has become waterproofed by the deposit of silt, the loss from absorption will be generally much less than that from evaporation; hence deep reservoirs, which present a comparatively small surface, will suffer less total loss on these accounts than will shallow ones. In the case of porous basins the loss by absorption

will be to some extent diminished by springs feeding the reservoir from its margin, which will generally be equally porous.

In the loss by absorption is included the loss by leakage under the dam, to reduce which a puddle trench is formed under the embankment. This loss should be comparatively very small.

The total loss will depend upon the time that these causes act on the storage, and will vary with the season. During an ordinary monsoon, as the air is then charged with moisture and is fairly cool, there would be comparatively little evaporation but for the fact that the wind for many days continues to blow with violence and thus causes a good deal of it to take place. In this period, however, the rainfall which falls on the water surface of the reservoir will be wholly impounded, and this will fully make up for the evaporation which then takes place, as in the run-off calculations it is not usual to consider the area of the tank separately from that of the rest of the catchment, from which the run-off is only a fraction of the rainfall.

In the cold weather evaporation will not be great, as, not only will the temperature then be low, but there will be generally an absence of high winds. In the hot weather, when the temperature rises, and hot, dry winds of some intensity may be expected, the loss will be at a maximum, and may amount to as much as 0·4[1] inch a day.

23. Observations of Loss by Evaporation and Absorption in Tanks. — The table on p. 24 shows observations of the amount of loss that have been made.

Probably in this case the compensation due to rainfall on the area of the reservoir during the monsoon months was not taken into account. The temperature throughout the

[1] Buckley's "Irrigation Works in India and Egypt," p. 65.

[1] AVERAGE DAILY LOSS IN THE RED HILL TANK, NEAR MADRAS.

Monsoon			Cold Weather.			Hot Weather		
Month.	No. of years observed.	Loss in inches.	Month.	No. of years observed.	Loss in inches.	Month.	No. of years observed.	Loss in inches.
July ...	4	0·33	November	1	0·27	March ...	5	0·26
August ...	5	0·32	December	2	0·13	April ...	5	0·30
September	1	0·38	January...	4	0·24	May ...	5	0·37
October...	1	0·27	February	2	0·24	June ...	4	0·30
Average	...	0·33	...	...	0·22	...	...	0·31

year is there more equable than it is in the northern part of India. The average daily loss throughout the year was found to be 0·28 inch and the annual loss, 8·39 feet; this large amount seems to indicate that there was much loss by absorption. The heaviest rainfall is in October and November, when it varies from 10 to 13 inches each month.

In the Ekruk tank, near Sholapore, Bombay Presidency, the loss from evaporation and absorption from April 17th to May 29th, the hottest part of the year, averaged 0·384 inch a day, while the similar loss, increased by some leakage, during November to March averaged 0·232 inch a day.

The loss[1] in tanks in Rapjutana is given by Mr. W. W. Culcheth as follows :—

Season.	Average daily loss in inches.		
	Evaporation.	Absorption.	Total.
During the irrigating season, October to March	0·15	0·05	0·20
During the hot season, April to June	0·29	0·17	0·46
During the rainy season, July to September	0·21	0·20	0·41
Average of the year	0·20	0·12	0·36

[1] Buckley's "Irrigation Works in India and Egypt," p. 66.

The soil was porous, and yet the loss from absorption was considerably less than that from evaporation; the former loss is irregular in amount, and it was probably difficult, as explained before, to measure it exactly. The total loss in the year is given as 9·77 feet, of which 6·15 feet was from evaporation and 3·62 feet was from absorption. These figures are unusually high.

In the Pashan tank, near Poona, the average daily loss, from evaporation alone, was in inches :—

October.	November.	December.	January.	February.	March.	April.	May.
0·25	0·19	0·14	0·17	0·14	0·17	0·27	0·38

During these eight fair-weather months, the total loss was 4·33 feet.

The usual allowance made in Bombay projects for all losses is 4 feet measured on the mean area of the tank, although, in some cases, it has been taken as low as 3 feet, and, in others, as high as 7 feet.

The following may be taken as the average monthly loss during the principal seasons of the year :—

Cold weather	2 ins.	—total for four months	8 ins.	
Hot weather	6 ins.	,, ,, ,, ...	24 ins.	
Monsoon	4 ins.	,, ,, ,, ...	16 ins.	
		Total for year	4 feet, or 48 ins.	

It will be seen from the above that for irrigation schemes it is not economical to store water for two years, as the loss from natural causes during so long a period will amount to a very large proportion of the total storage, and it would be far better, both for Government and the cultivators, to utilise in one season the full supply available, so as to lose only one year's loss by evaporation and absorption.

24. Observations of Loss by Evaporation and Absorption in Canals.[1]—As feeder channels have to be

[1] Buckley's "Irrigation Works in India and Egypt," pp. 68 and 69.

made in connection with some tanks, the following results of the observations of loss on certain canals may be given :—

(*a*) The Háthmathi Canal, Ahmedabad district, Bombay, with an ordinary discharge varying from 20 to 100 and a maximum discharge of 191 cusecs, is estimated to lose 50 per cent. of its discharge in the first 10 miles of its course.

(*b*) The Nirá Canal, in the Poona district, Bombay, with a head discharge of 455 cusecs and a length of 101 miles, has been gauged to lose 1 cusec a mile, or, altogether, 22 per cent. of the initial supply.

(*c*) The Muthá Canal, also in the Poona district, with a maximum discharge of 412 cusecs, loses from 0·8 to 0·9 cusec a mile.

(*d*) The Patna Canal, Bengal, having a bed-width of 69 feet and an average depth of about 6 feet, had a gauged loss of 40 cusecs in a length of 7 miles when it was new. The rate of loss has decreased greatly since the canal has silted.

(*e*) On the Bári Doab Canal, in the Punjab, after the canal had been open for some sixteen to eighteen years, the following gaugings were made[1] :—

Section of Canal.	Date of Observation.	Upper Gauge.		Lower Gauge.		Loss.		
		Mean velocity, ft. per sec.	Discharge in cusecs.	Mean velocity, ft. per sec.	Discharge in cusecs.	Discharge in cusecs.	Percentage.	
							Total.	Per mile.
Between Madhopur and Chandeki, 52 miles apart.	March 1st and 2nd	—	2,009	2·68	1,728	281	14·0	0·27
	May 26th and 27th	4·44	2,14	2·93	1,874	268	12·5	0·24
	May 28th and 29th	4·33	2,036	2·79	1,789	247	12·1	0·23
	June 9th and 10th	4·44	2·165	2·77	1,874	291	13·5	0·2
Between Hibban and Gandian, 63 miles apart.	July 31st ...	—	289·4	—	243·2	46·2	15·9	0·25
	August 2nd	—	384·3	—	298·4	85·9	22·4	0·35

[1] Buckley's "Irrigation Works in India and Egypt," p. 69.

Before taking such observations, the canal should be made to have a constant discharge for a few days before and during the time of the gauging, so that its perimeter may be thoroughly wetted, and then all distributaries should be closed and all sources of leakage should be cut off.

25. Comparative Cost of Storage.—In comparing the relative costs of storage of different schemes, the rates of storage per million cubic feet of "available capacity," and not of "gross storage," should be taken into account, because for the purposes of irrigation the storage below outlet level is useless. A more exact comparison would be made if also the amount of loss by evaporation and absorption were deducted from the available capacity. This is, however, never done, because its amount is not known exactly but has to be estimated; moreover, it will vary with the way in which the storage is expended, being least when the contents of the tank are drawn on extensively in the early part of the season.

This comparison of rates of storage is a most valuable one, as from it can be seen at once if the scheme is likely to be a remunerative one or not. In Bombay, where the rate of earthwork varies from about Rs. 1 ·· 2 ·· 0 per 100 cft. in small dams to about Rs. 1 ·· 8 ·· 0 in large dams, the following would be considered fair rates of storage for—

	Mill. cft.	Rs. per mill. cft. available capacity.
Small tanks storing less than	200	750
Medium tanks storing from	200 to 1,000	500
Large tanks storing over	1,000	400 to 300

26. Estimate of the Storage required for a certain Irrigable Area.—In some cases there may be a definite area to be irrigated, either wholly by the storage impounded in a reservoir, or, partly, by the natural discharge of a stream and, partly, by tank storage, and the amount of the

storage required has to be determined. A typical calculation is given in Appendix 3. In this case a full-supply storage is assumed at first and the effect of the draw-off from the tank is worked out. If this shows a deficiency of supply, the maximum amount of deficiency should be added to the assumed storage, and to this should be added a proper allowance for evaporation and absorption to give the required amount of storage. If the tank with the storage originally assumed shows an excess supply, the amount of the excess should be deducted from it before adding the allowance for evaporation and absorption.

27. Estimate of Expenditure from Storage—Duration of Supply.—The expenditure from a tank should be regulated as carefully as expenditure of money, for, otherwise, its contents may not suffice till the monsoon of the next year replenishes it. For this purpose an estimate of draw-off, &c., should be made, as given in the example in Appendix 4, and as explained in the notes appended to it, and the calculated rate of draw-off should not be allowed to be exceeded.

In Appendix 5 is given another estimate of the duration of a water-supply storage, and the method of its calculation is explained therein.

28. Preliminary Estimate of the Cost of a Storage Reservoir.—Before drawing out a scheme in detail, it is desirable to ascertain approximately if it will, or will not, be financially remunerative. The cost of a reservoir depends chiefly upon that of the dam, and the cost of this can be taken out roughly from the longitudinal section. To this have to be added the costs of the outlet, waste-weir, and land compensation, which can be taken out either approximately and independently, or with reference to the rough estimate of the dam.

The following table shows the comparative total costs of the subworks, &c., of twenty completed projects

with earthen dams in the Bombay Presidency (*vide* Appendix 1):—

1 Subwork.	2 Total Cost in 20 Projects. Rs.	3 General Percentage Cost of Subwork of Total Cost.	4 General Percentage Cost of Subwork of Cost of Dam.
1. Dam	33,70,028	78·05	100·00
2. Outlet	2,33,473	5·41	6·93
3. Waste Weir	4,07,535	9·44	12·09
4. Land Compensation ...	3,06,471	7·10	9·09
5. Total	43,17,507	100·00	128·11

The total cost of the individual projects varied from Rs. 30,732 to Rs.9,39,897: 7 works cost under Rs.1 lakh each; 9 works, from Rs.1 to Rs.3 lakhs each; and 4 works cost over Rs.3 lakhs each.

In the table, col. 3 gives the general percentage cost of each sub-work of the total cost of the projects, and col. 4, that of the total cost of the dams alone.

The capacity of the tank having being ascertained, the approximate rate of storage per million cubic feet can be thus found out. The rate due to the canal and distributing works will then have to be added, the total cost of these being deduced from that of existing projects. The approximate net return from one million cubic feet of storage can be estimated as shown in Appendix 7, and it then can be seen if this is sufficient to produce a proper return on the expenditure.

V. The Irrigational "Duty" of Water.

29. The Amount of Storage dependent upon the "Duty."—In order to ascertain the quantity of storage required in a tank to irrigate a certain area, or, *vice versâ*, the area which can be irrigated by a certain amount of storage, it is necessary to know first what is the

irrigational "duty" of the different classes of crops. The "duty" is the acreage which can be brought to maturity by the constant flow, during the season the crop is on the ground, of one cubic foot of water a second.

Appendix 6, Table I., gives the duties which have been obtained during certain non-famine years on certain irrigation works in India supplied by tanks and canals. It will be seen from this that the duties vary greatly, and that the main general result is that they are smaller on works dependent upon tanks than on those that are served by canals without artificial storage. This is due to the fact that the country in the neighbourhood of tanks is generally of a more irregular nature, and the area under irrigation by them is less concentrated, than in the alluvial tracts irrigated from canals without storage.

Appendix 6, Table II., gives the duties which have been obtained during certain non-famine years on certain irrigation works in Bombay (Deccán). These also vary considerably owing to local peculiarities; the main general result is that the kharif duties are higher than the rabi ones; this is probably due to the rainfall, which is practically limited to the former season.

To determine the amount of storage required, it is best to ascertain the duties obtained from constructed works in the neighbourhood, but, where this information is not available, it will be necessary to estimate, or assume, them. As a general estimate it may be assumed that under tanks the following will be the average duties when the irrigation is fully established:—

Rice, 40; monsoon dry crops, 160; rabi crops, 120; perennial crops, 100; and hot weather crops, 60 acres per cusec (*vide* Appendix 7 (1)).

30. The Proportion of Irrigable Area to Culturable Area under Command.—Owing to the expense of, and

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soil exhaustion produced by, irrigation, it is not likely that the whole culturable area commanded will be irrigated. Appendix 6, Table III., shows the actual results obtained on irrigation works in the Bombay Presidency (Deccán). During recent years the percentage of the irrigated to the culturable area under command has been 33·9, but, as on these works the land at the tail of the systems has generally not yet been brought under command, and, as the proportion irrigated there will be less than it will be at the head, it will be safer to assume that not more than one-quarter of the culturable land under command will be eventually irrigated annually by a tank.

On the great canal systems of Northern India, in certain tracts where water-logging and consequent salt efflorescence have to be prevented, the percentage of irrigated area has to be restricted, rather than encouraged. It has been decided that, in the area served by the Chenáb Canal,[1] as long as the spring level is more than 40 feet below the surface, the limit of the percentage of irrigated to culturable area may be 50, but, as the water level rises, the limit is to be reduced, until when the subsoil saturation plane is within 10 to 15 feet of the surface, irrigation is to be stopped altogether. On lands irrigated by tanks, where the surface of the ground is generally not uniform in level and is intersected by numerous well-defined drainage lines, such water-logging is rarely to be feared or guarded against.

31. The Proportions of different Crops under Irrigation.—To estimate the revenue probable from the work it will be also necessary to take into account the proportions in which the different crops are likely to be irrigated, as the quantities of water required for them and the

[1] "Recent Developments in Punjab Irrigation," by Sidney Preston, C.I.E., 'Journal of the Society of Arts," Vol. l., No. 2,584, May 30th, 1902, p. 607.

assessments on them vary. The factors which determine these proportions are :—

(*a*) The suitability of the soil and climate ;
(*b*) The sufficiency of the water-supply ;
(*c*) The market prices of the produce.

Of these (*a*) are fairly constant; (*b*) should, as far as practicable, be made constant by good arrangements ; but (*c*) is beyond local control, and variations due to it must be accepted as not preventable.

Appendix 6, Table IV., shows the average results obtained during certain years on certain works in Bombay (Deccán). It is best to depend upon those of neighbouring works situated under like conditions to the project under preparation, but, where such works do not exist, it will be fairly correct to assume that the following percentages of crops will be irrigated :—

Perennial and hot weather	20	Total 100.
Rabi	20	
Monsoon dry	40	
Eight months	20	

(*Vide* Appendix 6, Table IVA.)

32. The Rotation of Crops.—The above percentages indicate that a system of quinquennial rotation is practised under the recently established Bombay irrigation tanks, namely two years of monsoon dry to one year of each of the other classes of crops. This is the lowest form of rotation, while triennial rotation—perennial, eight months, and rabi crops—is the highest. Rotation is necessary to secure restoration of fertility to the soil, which would, without it and artificial help from manures, soon become exhausted if constantly irrigated by tank-water, which is practically devoid of fertilising silt. The rotation should, however, be as high as possible, for the revenue chiefly

depends upon the proportion of the perennial crops to the whole area under irrigation (*vide* Appendix 7 (3)).

VI. Tank Basins.

33. The Proportion of the Submerged Culturable Area to the Area Irrigable by the Project.—The storage of water in a tank involves the submergence of the land in its basin, and this, in the majority of cases, renders it unculturable. If the catchment area is rocky, the bed of the tank will in time become covered with a layer of sandy material, which is unsuited to cultivation. If, however, the drainage area is of soil, the bed will be covered eventually with fertile silt, which may render it better fitted for agriculture than it was originally; but, even in this case, it will be at first too damp to plough, and may subsequently cake in drying, so that crops cannot be grown on it. Moreover, the bed may not be laid dry until the cultivating season has passed. It is generally only shallow tanks, whose contents are expended during the monsoon, which can be cultivated, and that in the rabi season. It may be thus said that the construction of a reservoir usually throws the submerged area permanently out of cultivation: much of this owing to its situation may have been extremely fertile originally. It is therefore necessary that this area should be as limited in extent as possible, for the area to be served by the work may not always be irrigated, and, if the submerged area bears a large proportion to it, there may not be, in a series of years, a substantial increase of the area cultivated on that which existed before the construction of the project. This argument, of course, applies to culturable land, or land which can be cultivated, as waste lands, except so far as they are useful for grazing, need not be taken into account

in this connection. Speaking generally, the culturable area in a tank basin should not exceed one-fifth of that which can be irrigated by the work.

For this reason deep tanks, which occupy little land compared with the quantity of their storage, are to be preferred to shallow ones. The former have the added advantage that, as they have a comparatively small surface area, the loss by evaporation from them will be a minimum one. In this respect the best storage basins are those which open out immediately behind the dam, as this part of their area has the greatest depth of water over it. The slopes of the country near the dam should be preferably flat ones, but it is not of so much importance that those at the head of the tank should be gentle, as there the depth of water will be shallow.

34. The Retentiveness of the Tank Basin.—It is necessary for economical storage that the bed of the basin should be of a retentive nature, so that the loss by absorption may be minimised as much as possible. It is true that the longer the tank is in use, the more will it be waterproofed by the deposit of silt on its bed; but this silt is itself naturally porous, owing to its recent deposition and want of consolidation by pressure, and it is therefore best to start with naturally retentive substrata. Moreover, if a large area of the tank bed is annually laid dry, the overlying silt will absorb an appreciable amount of storage. The most retentive basins are those with deep, clayey, alluvial beds, and the least retentive are those with vertically stratified and fissured strata.

35. The Extent and the Utilisation of the Land to be Acquired.—When acquiring land for a reservoir, it should be taken up at least to high-flood level, as otherwise the submergence of marginal crops during floods may lead to their destruction and to claims for the payment of compensation. In fact, if the land is cheap, it should be

acquired originally to a slightly higher level to permit of the future increase of the storage being obtained without having subsequently to pay extravagantly for the marginal land, which may, by that time, have become increased in value, either by natural circumstances, or by advantage being taken by its proprietors of the necessity for its purchase. The limits of the acquired land should be at once permanently marked by boundary stones, whose positions, with reference to the ordinary field survey marks, should be off-setted and recorded on the maps and in tabular statements.

In order to render the acquired marginal land as productive as possible, it should be planted out at once with trees, as the young plantations can be watched best while the construction of the works is in progress. Bamboos and trees which flourish in damp situations should be planted in the zone next full-supply level, and fruit-bearing trees, which will produce an annual revenue, should be reared in solid blocks near the dam, where they can be most easily guarded. The rest of the area should be devoted to less valuable trees, which can be cut down periodically for timber or fuel.

Tank irrigation exhausts the land very much, as the supply from storage reservoirs is clear and devoid of the fertilising silt which is contained in the water of canals fed by perennial rivers flowing through alluvial soils. In order to develop such irrigation it is necessary that there should be a plentiful supply of manure, and, as in India manure is much used for fuel, it is most desirable to establish fuel plantations to set this artificial soil-stimulant free for agricultural purposes. The tank plantations recommended will serve usefully for this purpose. Other plantations can be formed with advantage on waste lands commanded by the canal.

VII. The Silting of Reservoir Beds.

36. The Necessity and the Methods for the Reduction of Silt Deposit.—As explained in paragraph 34, the silting of reservoir beds has the advantage of making them more retentive, but it has also the disadvantage of reducing the storage capacity, which is a result far outweighing the former one. It is not economically practicable to remove this silt by any artificial means, as the cost of storage is very small compared with that of any system of excavation. For this reason it is desirable to prevent the accumulation of silt by choosing a bare or rocky catchment, which will not produce much deposit, and one with a certain replenishment, which will not necessitate an increased catchment area to insure a full storage in bad years, as, under ordinary conditions, the water which enters a tank will deposit practically all its silt in it.

The most heavily silt-laden floods are those which come early in the season, as they carry down with them the soil which has been desiccated during the fair season. The best way of dealing with the silt question is to pass these early discharges out of the tank as rapidly as possible by means of outlets with large discharging power. The amount of replenishment due to these floods can be, however, foregone only in cases where the subsequent run-off is sufficient to ensure the filling of the tank. Such reduction of silt can be effected best when it is necessary to impound only the clear-water flow subsequent to the close of the monsoon: an example of this treatment is furnished by the large Assuán reservoir on the Nile.[1]

Probably the simplest method of removing part of the silt which is practicable in India is to plough the dry bed of the reservoir as early as possible so as to loosen and desiccate

[1] "Minutes of Proceedings, Inst. C.E.," Vol. clii., Paper No. 3361.

its surface and allow it to be carried by the action of the rain into the pool at the bottom of the tank basin, and thence to be removed by the flow produced by the discharge through the outlet. The deposit for a short distance round the margin of this pool may be also moved closer to it by means of scrapers.

37. The Classification of Silt.—The deposit in reservoirs may be divided into three classes:—

1. Heavy detritus, consisting of stones, pebbles, and large sand ;

2. Heavy silt, consisting of fine sand and earthy matter which is deposited in a few days ;

3. Fine silt, consisting of earthy matter which is held in suspension for several days.

The first class is deposited directly the velocity of the inflow is checked by its entry into the reservoir pool, and will be thus found along the beds of the feeding streams, and particularly at the head of the tank. If the outlet is kept open for some time in the early part of the monsoon and so as to give a large discharge, the deposits of the previous years may be moved gradually forward towards the dam, and a part of them may be eventually swept through the outlet, but the proportion thus disposed of will be generally very small compared with the total quantity deposited.

The second class, being originally distributed uniformly throughout the contents of the reservoir, will be deposited in direct proportion to the depth of the storage, and its deposit will be thus thickest in the deep parts of the tank nearest the dam, and will be thus in the most favourable position for being passed out through the outlet. Being, however, of a tenacious and compact nature, it will resist removal unless loosened artificially, as by ploughing, as mentioned above.

The third class will, similarly to the second one, but

more slowly, be gradually deposited, and chiefly near the dam, and can be similarly dealt with. In both these classes the amount which can be removed by being passed out through the outlet will be only a small portion of the whole deposit. As the water stored in the reservoir will be in a state of quiescence for several months, practically most of the silt which enters the tank will be deposited in it. It will be only the light sediment held in suspension for some days which will be carried over the waste-weir when that is flowing.

38. The Proportion of Silt to Water.—The total quantity of silt brought into a tank will depend upon the nature of its catchment. The following table[1] gives the results of some observations of the weight of silt which is carried by rivers :—

1	2	3	4	5
River.	Experimenter.	Approximate Velocity, ft. per second.	Proportion of Silt to Water by Weight.	Remarks.
1. Mississippi	Humfreys and Abbot	4	$\frac{1}{575}$	
2. Rhône ...	Gorsse and Subours	8	$\frac{1}{45}$	
3. Po	Lombardini ...	—	$\frac{1}{300}$	
4. Vistula ...	Spittel	10	$\frac{1}{48}$	River partly frozen.
5. Rhine ...	Hartsoeker	—	$\frac{1}{100}$	
6. Nile ...	Letheby	3	$\frac{1}{688}$	Equal parts of sand and mud.
7. Rio Grande[2]	Anson Mills ...	10	$\frac{1}{190}$	By volume. Average of 118 samples.
8. Indus ...	Tremenheere ...	5	$\frac{1}{237}$	Equal parts of sand and mud.
9. Ganges ...	Medlicott	10	$\frac{1}{780}$	At head of Ganges Canal.

These large rivers flow through fertile areas, and consequently carry much silt. Generally, reservoirs will be situated in hilly and less fertile tracts, and, in their case, the weight of silt will probably be less, and might on an average be taken as $\frac{1}{1000}$ of the weight of the water.

[1] Buckley's "Irrigation Works in India and Egypt," pp. 36 and 37.
[2] Schuyler's "Reservoirs," p. 361.

The silt when first deposited will be of a light, bulky nature, but, in course of time, will become more compact, and thus will occupy less volume, and eventually will attain a density nearly equal to that of ordinary soil. Taking the weight of soil as about 100 lbs. per cubic foot, the final bulk of the silt will be about two-thirds of that of the volume of water to which it was originally equal, and thus the volume of silt deposit, compared with that of the water in which it was suspended, will be about two-thirds of the above proportions by weight.

39. The Determination by Cross-sections of the Amount of Silt Deposited.—There are three ways in which the amount of silt which is being deposited in a tank may be determined, by :—

1. Section lines across the reservoir bed ;
2. Silt boxes in the reservoir bed ;
3. Direct measurement of the inflow into and outflow from the tank.

In the first system, cross-section lines are ranged across the tank bed at intervals and have their ends permanently marked by masonry pillars about 5 feet high, so as to be visible from each other, and with their distinctive numbers and levels engraved on their capstones. Where it crosses every 10 feet contour the line is marked with header-stones, also engraved with their distinctive numbers and having their surfaces at original ground level. Iron index rods projecting 3 feet above ground-level are fixed close to them, so that their positions may be readily ascertained even after the stones are covered by silt. The original sections are plotted, and they are relevelled and replotted at intervals of a few years. Although this system will give the exact depths of deposit varying from point to point on the section, it has the objections that it will take several years to ascertain any great difference of level, during

which the record may be lost, and also that the number of lines has necessarily to be small, so that the total deposit over the whole bed may not be exactly ascertainable from them.

40. The Determination by Silt Boxes of the Amount of Silt Deposited.—In the second system, wrought-iron boxes, about 2 feet by 2 feet by 1 foot with closed bottoms and open tops, are placed at intervals along section lines ranged across the tank bed. Each box has fastened to it a substantial chain by which it can be lifted, and to this is fastened a light wire attached to a large float, so that the position of the box can be ascertained when the tank is full. When the boxes have to be examined, they are carefully lifted and the silt either measured or weighed. The objections to this system are:—the floats may break loose in stormy weather; the boxes are difficult to lift when in deep water, and, possibly, their contents may be upset when they are being lifted; and the number of lines and of boxes in them have to be too few to determine accurately the total amount of deposit over the whole bed of the tank.

41. The Determination by Observations of Inflow and Outflow of the Amount of Silt Deposited.—In the third system, samples of the inflowing water are taken, daily when the rate of inflow is steady and at more frequent intervals when it varies, and the proportions of silt to water, either by volume or weight, are ascertained by passing the whole samples through filters and collecting the silt on them. Until the waste-weir comes into action, the increase in the contents of the tank can be readily ascertained from the tank capacity table. While the waste-weir is flowing, samples of its discharge have similarly to be taken, the amount of the discharge has to be calculated, and the amount of silt carried away can then be

determined and deducted from that brought into the tank. By this system a very fair approximation to the actual amount of silt deposited can be ascertained, but the way in which the deposit is distributed over the bed cannot be thus found out; this is, however, not a matter of much importance. The objections to this system are that it involves a large series of observations, and also, that the samples of water taken may be not truly representative of the whole discharge. Still, it affords the quickest and most exact form of silt measurement.

42. The Level at which the Outlet Sill should be Placed.—It does not require any system of silt measurement to prove that the amount of deposit from ordinary catchments is very considerable, as after a few years this is most perceptible, and, even in twenty years, in many cases has been sufficient to diminish seriously the storage contents of existing reservoirs. It is for this reason not desirable to place the outlet sill at a very low level, for, if this is done, the tank bed will soon silt up above it. As a rule it will be advisable to fix its level, so that the original storage below it will be about 10 per cent. of that of the whole reservoir. The original contents of twenty existing tanks in Bombay above and below their outlet sills were 10,108·120 and 779·834 mill. cft. respectively, or 92·84 and 7·16 per cent. of their total storage capacity. Deducting one tank, where the storage below outlet sill was proportionately very small, viz., 20 mill. cft., compared with that above it, 3,310 mill. cft., the storage capacity below outlet sill of the remaining 19 tanks was originally 10 per cent. of the total storage.

As the bottom contours are of small area, this allowance for silting will entail the raising of the outlet sill some feet above the lowest bed level of the dam. A further raising to any extent of the outlet sill is not desirable, as it will

lessen considerably the storage to be drawn upon for a number of years before the silt occupies the space below it, and will thus lead to a reduced area of irrigation and to a loss of return on the capital expenditure.

43. The Advantage of Earthen Compared with Masonry Dams in respect to the Silt Difficulty.—Although it is economically not possible artificially to remove much of the silt deposited, the original storage, in the case of an earthen dam, can be easily restored when necessary, and at a cheap rate, by raising the dam and the full-supply level. This is one of the chief advantages of an earthen dam over a masonry one, for in the case of the latter, raising the levels is not so easy a matter unless it has been arranged for originally by providing a wider section to permit of it. If, however, this has been allowed for, it means that a considerable amount of the capital cost will be infructuous until the time for raising has arrived.

VIII. Survey and Investigation Work.

44. Survey for the Works.—The first survey work to be undertaken is in connection with the proper location of the dam line. A trial centre line should be first set out and contours should be run at convenient intervals, say 4 feet apart, over the whole area which is likely to contain the final line. After plotting this work to a large scale, the most economical line can be ascertained by trial in office, *i.e.*, by setting out different alternative lines on the plan, and by estimating roughly the costs of dams constructed on them.

After the final line has been thus selected, it should be levelled over, points in the longitudinal section at every 100 feet being noted and also those where the ground slopes change. Longitudinal and cross-sections of the waste-weir and outlet channel lines should be then taken

and the plan completed by running contour lines at 2 feet apart vertically over the whole area likely to be occupied by the works. From such a plan it will be easy to make any changes in the design which may be found necessary when working up the project. (*Vide* Plate 4, Fig. 1.) The plan should be extended by ordinary survey so as to include the whole area likely to be occupied by temporary works.

The areas from which the different soils required for construction are to be obtained should be next surveyed and plotted, small trial pits being excavated to test their depths. From this plan a tabulated statement of quantities, leads and lifts can be prepared in order to ascertain the earthwork rate to be allowed for the construction of the dam. Similarly, the sites from which all other requirements, such as water, sand, lime, fuel, and building and pitching stone are to be procured, should be ascertained and the quantities available roughly determined so as to arrive at the proper rates to be allowed for these materials.

45. Survey of the Capacity of the Tank Basin.—After this has been done, the tank basin should be carefully contoured, the contours being usually taken at vertical intervals of 10 feet apart in the case of large basins. In the case of small tanks, cross-sections may be run at intervals across the basins, and the contours can be then plotted from them. It is as well first to fix the contour points by level and then to triangulate the areas comprised by them so as to ascertain their extent correctly. Where necessary, additional lines should be measured as tie lines. As village maps are not very accurate, the contour plan should be prepared independently of them, but the contours may be also laid down on these maps with reference to field boundaries, so that the levels of outlying temporary works may be ascertained at any time, and also, so that the extent of land to be acquired may be marked

on them. The limits to be acquired should be defined permanently by stones, on some of which bench marks should be engraved. These stones should, as a rule, be fixed on field boundaries, and their distances from field marks should be measured, or off-setted, and recorded both on the plan and in a tabulated statement.

46. Rainfall and River Discharge Observations.— Rain and river gauges should be established as soon as the investigation of a project is determined upon. In regard to rain-gauges it has to be remembered that a single gauge may not be representative of the rainfall on the whole catchment, and will certainly not be so if it is of any extent and if its physical character is of a variable nature. (*vide* paragraph 14). It is best, therefore, to fix a rain-gauge to ascertain the rainfall of each constituent area with a variable fall, and especially should this be done during the years in which the construction of the tank is in progress, as then there will be superior establishment available for checking purposes. With a series of such numerous observing stations continued for a number of years, it will be possible to ascertain the relative rainfall on the different constituent areas of the whole catchment, and, when this has been found out, the number of stations can be subsequently reduced greatly and the results from the diminished number applied to the whole catchment. There are several forms of automatic recording rain-gauges, but these are not usually employed in India, although in such cases they well might be used for checking purposes.

The rainfall observations should be always supplemented by ones of river discharge, for these will give the actual amount of run-off and will take into account all the factors producing it. The river-gauge should be established just below where the outfall of the waste-weir enters the stream, so that observations of it may be continued, if

necessary, after the completion of the work. It should be permanently set out and cross-sections of it taken, so that from them a tabular statement of the different discharges at different gauge readings may be made out and utilised in the future. For this purpose a reach of the river should be selected where the bed is uniform, both in longitudinal and cross section, and where the velocity of flow is regular and not extremely great even during ordinary floods.

IX. The Utility of Tanks.

47. The Utility of Tanks in Times of Scarcity and Deficiency of Rainfall.—There is an idea generally prevalent that, by the construction of numerous tanks, the country may be eventually protected from the worst effects of famine, but it is questionable if this is a correct one. It has been explained in paragraph 7 that, in the case of reservoirs situated in the plains and in the area of uncertain rainfall, the catchment areas, however large, in such seasons may be unproductive of run-off sufficient to fill them. If the run-off then were sufficient, this would imply that there would be a still larger quantity of rainfall in order to produce it, and this amount of rainfall probably would suffice to prevent entire scarcity. The real benefit of tanks constructed in such areas is that, when the rainfall is deficient on the whole, or is so irregularly distributed as not to be capable of bringing crops to maturity, their storages will be able to supplement it and to permit of the growth of the crops. There are many more years of deficient and irregular rainfall than there are ones of total scarcity, and it is during these former years that tanks will be of substantial benefit and will fully justify their construction.

While the failure of even the largest tanks constructed

in the plains may be expected in years of great deficiency of rainfall, the case is different with those which are situated in the ghâts, which have an unfailing rainfall. Although the rainfall there is also liable to great variations, still even in the worst years there is in such localities sufficient to cause run-off, which is all the greater owing to the steep, hard and bare slopes which usually characterise the surface of such catchments. It is therefore highly desirable that all such sites should be utilised first, as they are likely to produce most revenue and benefit to the people, the works in the plains being postponed until it is necessary to construct them for the purpose of employing relief labour.

48. The Comparative Utility of Small and Large Tanks.—A small tank, under the most favourable circumstances, will irrigate only a small area, and this, considered with reference to the whole country, will be of inappreciable benefit. Moreover, in times of scarcity of rainfall, the replenishment of such tanks is likely to be deficient or totally to fail, and they will be thus practically, or wholly, useless. The maintenance of tanks having a storage of less than 50 mill. cft., if attended to by Government, will be relatively costly, and it will be more difficult to arrange for the supervision of numerous small works than of a few large ones having the same total storage.

The advantages of small tanks are that they will utilise small catchments which might otherwise be wasted, and will provide for the irrigation of isolated areas which might not be otherwise developed. If, moreover, they are not too large, and each can serve only one or two villages, their management may be entrusted to the cultivators, Government attending only to the maintenance of the works.

Large tanks, on the other hand, will have larger and more unfailing catchments; will be capable of irrigating large areas which will sensibly affect the productive capacity of

the country as a whole; their rate of storage will be cheaper; and their maintenance will be comparatively cheap and their supervision easy. Owing to the variety of the interests they will have to serve, their management will have to be arranged for entirely by Government.

In regard to mitigating the effects of scarcity, the proper programme therefore seems to be to construct in the first instance large tanks with unfailing catchments, then those with less certain ones, and finally small ones for the benefit of isolated areas.

49. Revenue Prospects of Tanks.—Owing to the low rates charged in India for irrigation, to the excessive loss of water by evaporation, &c., in the storage reservoir, to the large quantity required to bring the crops to maturity, and to the great cost of storage works, it is doubtful if at first tank irrigation will be ever remunerative; but, as time goes on and the value of the produce increases, the rates may be increased proportionately, and the revenue results will consequently improve. The benefit to the country as a whole has also to be taken into account: in normal years such works will produce crops which will add to the wealth of the cultivating classes, while in an abnormal year works with an assured supply may easily mature crops the value of which will be nearly equal to their capital cost. If an irrigation work pays a little more than its working expenses during normal periods, it may, from this point of view, be accepted as a scheme financially sound in regard to the country as a whole, although far from being a directly remunerative one to Government. If expenditure on relief during times of famine is devoted to the construction of such works, instead of being incurred on works of only temporary utility, each successive famine will thus permanently enrich the country and render it better able to withstand the effects of future scarcity.

It is useless to expect from tank irrigation the fine returns which are derived from the large canals which are supplied by the great perennial rivers and do not require expensive storage works, as the natural conditions are unfavourable to the former and are favourable to the latter. The proper way of regarding tank schemes is to remember that they are the only ones available for the lands they serve, and that it is the duty of Government to develop the country so far as their finances will permit.

50. The Suitability of Tank Works for the Employment of Famine Labour.—The construction of tanks affords the most suitable work for famine labour. The quantity of work available on them for unskilled labour is so great that it will afford employment for large numbers during the whole time of scarcity. There is not thus any necessity for the expense of having to move the people frequently and to make numerous camps and other arrangements for them as is necessary in the case of such less concentrated works as roads and railways. Moreover, the nature of the work is suitable for the agricultural population, which will comprise the large bulk of those who come for employment. The supervision of the people will be easier, and will be arranged for more cheaply, than it can be on any other less concentrated class of work.

One objection to tank works is that they cannot be completed in the one season during which alone it is likely famine conditions will prevail. However, recent experience has shown that distress may last for more than a single season, and, even if it is not subsequently acute, it is fairly certain to exist in a more or less pronounced degree for some time during which the provision of employment is desirable in order to enable the people to recuperate both physically and financially. The construction of tank works will not interfere with, but will be in addition to, their means of

ordinary employment, whereas, if in times of scarcity they are put on such works as metal breaking for roads, many will be deprived in subsequent years of their ordinary means of subsistence.

It is, of course, necessary that a well-considered programme of works should be arranged before a famine has to be dealt with, so that during it the construction of projects with poor prospects may be avoided.

CHAPTER II.

THE DAM EMBANKMENT.

I. Various Kinds of Dams.

51. Classification of Dams.—The different descriptions of dams may be classified thus :—

1. Purely earthen embankments ;
2. Earthen embankments with stone toes ("compound dams") ;
3. Masonry dams ;
4. Composite dams ;
5. American types of dam.

52. Earthen Embankments.—A purely earthen embankment may be formed in the following ways, with :—

(*a*) A puddle wall at the centre, or, on the water slope, or, in some intermediate position ;

(*b*) A hearting supported on each side by more stable material (this hearting is practically a very wide puddle wall) ;

(*c*) A homogeneous section, without a puddle wall.

In English practice (*a*), having the central puddle wall, is the type generally adopted ; in recent practice in Bombay (*b*) has been adopted ; while (*c*) was constructed there for the earlier dams, and, with an important modification of these examples, to be described hereafter, (*vide* paragraph 80), is that which is now recommended.

Earthen embankments with stone toes ("compound dams") may, in respect of the upper part, be constructed in any of the ways adopted for purely earthen dams ;

they differ from them, in respect of the lower part, in having stone toes to support the base when the work has to be carried to a great height.

53. Advantages of Earthen Embankments.—These two classes of embankment are the ones which will be dealt with in this chapter. The advantages they possess over other types are:—

(1) They do not require such expensive and solid foundations.

(2) The materials for their construction being necessarily close at hand, (or they would not be economically practicable), labour on them can be concentrated and easily supervised, and they are therefore peculiarly well adapted for the employment of famine labour.

(3) They can be raised from time to time to meet the demand for more water, or to restore the deficiency of storage due to the silting up of the reservoir.

(4) They can be constructed quickly and by unskilled labour.

(5) They are the cheapest type, and, with suitable design and construction and proper precautions, can be made of any height which is ordinarily required in practice.

54. Masonry Dams — Composite Dams. — Masonry dams are the most stable of any form of dam. They, however, generally require deep and expensive foundations, and their construction is slow and expensive. They cannot be raised beyond their originally designed height unless their section has been made sufficiently wide at first for this purpose, a proceeding which involves the locking up of infructuous capital until the increase in height has been carried out. They are the best form for large and deep water-supply reservoirs, where risk of failure must be avoided at all costs.

Composite dams are ones in which part of the length is

formed as an earthen embankment and part as a masonry dam. In respect of costliness they are a mean between the two forms of which they consist. They are best adapted to sites where the river crossings are deep and the flanks are on high ridges. The junction between the masonry and the earthen embankment has to be very carefully made in order that there may not be a risk of water finding its way through the structure at this point.

55. American Types of Dams.—Schuyler[1] gives the following classification of "rock-fill" dams, *i.e.*, those having an upstream :—

1. Facing of two or more thicknesses of planking ;
2. Facing of asphalt concrete laid on a sloping dry wall;
3. Facing of Portland cement concrete laid on a dry wall ;
4. Facing of masonry built vertically and backed with earth which is covered on the downstream side with blocks of stone laid in mortar ;
5. Facing of steel plates laid on the upstream slope of a dry, hand-laid wall ;
6. Facing of earthen embankment ; or, having a
7. Central core of steel plates without hand-laid facing walls.

Other types of American dams are :—

8. "Hydraulic-fill" dams ;
9. Earthen embankments with masonry core walls ;
10. Earthen embankments with steel-plate core walls.

56. "Rock-fill" Dams.—The rock-fill dams are cheap to construct, but depend for safety on the facing, for, if that is injured, or decays, the great pressure of water in the reservoir would rapidly disintegrate the hearting and carry away its constituent blocks, no matter how large.

[1] Schuyler's "Reservoirs," p. 1.

There have been several instances of the failure of this class of dam, and the bursting of the Gohna Lake,[1] which was formed by a large slip blocking the Biráhi-gangá, a tributary of the Alaknandá, (a principal affluent of the Ganges), in a narrow valley in the Himalayas, by a natural rock-fill dam of immense thickness (2,000 feet wide at the top, 11,000 feet at the base, and about 900 feet high), shows that this form of construction cannot be depended upon, so far as the hearting is concerned. It is true that the primary cause of the breaching of this mass was its being overtopped by the reservoir, but it early showed signs of failure due to percolation, which percolation eventually amounted to 350 cusecs.

Most of the facings described above appear to be of a very temporary nature, and sufficient time has not elapsed since they were constructed thoroughly to test their permanency.

When steel-plate cores are used, they are sometimes protected by casings of asphalt concrete, 4 inches thick, but these have been known to slip off[2] during construction, and the steel plates themselves to buckle[3] by expansion. In[2] one example the steel-plate was $\frac{5}{16}$ inch thick for the lowest 20 feet, $\frac{1}{4}$ inch for the middle 20 feet, and $\frac{3}{16}$ inch for the top 28 feet.

57. "Hydraulic-fill" Dams.—The hydraulic-fill dam[4] is formed by sluicing an earthen area with a hydraulic jet and by conveying the resulting silt-charged stream on to the site of the dam, where the earth is deposited between shallow side banks, with the finer material as the hearting and the coarser as facings, and the clear

[1] Selections from the "Records of the Government of India in the P. W. D.," No. cccxxiv.
[2] Schuyler's "Reservoirs," p. 64.
[3] *Ibid.*, p. 22.
[4] *Ibid.*, pp. 76—116.

water is allowed to pass off. This form of construction requires :—

1. An abundance of water at the proper elevation to provide a sufficient "sluicing head";

2. An abundant deposit of suitable material for forming the dam conveniently situated at each flank, and high enough above the top of the dam to permit of the flow of the material required to complete it.

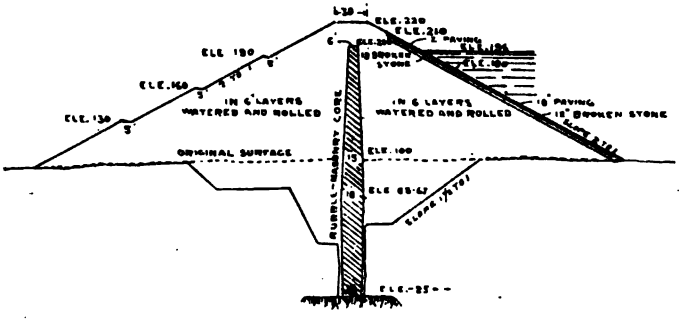
Where these conditions exist, the construction of the dam is both rapid and cheap, but the absence of water at a sufficient elevation will prevent, in India, the adoption of this form of dam making. This method appears to be a fairly sound one; the objections to it are that the layers are stratified and are deposited very wet, and thus much infiltration of water into the dam may occur.

58. Earthen Embankments with Masonry Core Walls.—American engineers are greatly in favour[1] of the construction of masonry core walls for earthen dams. One section[2] which has been recommended is 4 feet or 5 feet thick at foundation level, 8 feet thick at ground surface level, and 4 feet thick at the top. Another was only 2 feet thick, uniformly from top to bottom. These sections appear too thin to cut off infiltering water under great pressure, and the first one, although superior to the second in this respect, appears to have been copied from puddle wall practice, and is not of a stable form for masonry. A much better section proposed for the New[3] Croton Dam, New York (Fig. 1), had the trench portion 18 feet thick throughout, for 89 feet from foundation level to within 36 feet of ground level, and was thereafter battered uniformly on both sides for 136 feet up to its top, which was 6 feet wide and

[1] Schuyler's "Reservoirs," p. 281. *Engineering News*, Vol. xlvii., p. 153, Feb. 20th, 1902.
[2] "Minutes of Proceedings Inst. C.E.," Vol. cxxxii., p. 254.
[3] *Ibid.*, p. 267.

20 feet below the top of the dam, which was 245 feet above the foundation of the core wall. Even this thickness of core wall does not seem sufficient to stop entirely the filtration due to the enormous pressure, nor to be able to resist any unequal settlement of the earth on the two sides, which is the great danger to be feared in this form of construction. To prevent this unequal settlement occurring

FIG. 1.



CORE WALL OF NEW CROTON DAM

and producing lateral and unsupported pressure, the earthwork should be carried up uniformly on the two sides of the core wall.

The following advantages[1] are claimed for a masonry core wall :—

(1) If founded on an unyielding, impervious stratum, it forms a perfect cut-off in the centre of the dam, preventing water which percolates into the upstream slope from reaching the downstream one. (Against this it may be said that masonry, especially if thin, is not absolutely water-tight when subjected to great water pressure.)

[1] "Minutes of Proceedings Inst. C.E.," Vol. cxxxii., pp. 267 and 268.

(2) It cannot be washed out, as a puddle wall may be, should a leak through it be formed; in fact, such a leak through the masonry is more likely to silt up than to be enlarged.

(3) It separates the dam into two distinct portions, an upstream one, which should be made as water-tight as possible, and a downstream one, which should be made as stable as possible. If these two kinds of earthwork joined each other without the interposition of the core wall, cracks might occur in the centre of the dam on account of differences in their settlement. (Such different settlements might, however, still happen if the core wall were built, and might then lead to its fracture.)

(4) It enables the outlet culvert to be carried through the dam with perfect safety.

(5) It enables the outlet tower to be replaced by a "dry well" tower, built upstream of, and in connection with, the core wall, thus dispensing with the need of an outlet tower bridge (*vide* Chapter IV., paragraph 207).

(6) It gives an earthen dam greater strength to resist the erosive action of water passing over its top. (This topping of a dam should be prevented by sufficient waste-weir discharging power, and water should not be allowed to rise to within several feet of the top of the dam.)

On the whole it appears that the masonry core wall is superior to the puddle wall, but its expense would be probably prohibitive in India.

To sum up, it may be said that in India rock-fill dams would be considered to be dangerous to construct in view of their apparently temporary nature: hydraulic-fill dams are not practicable on account of the absence of high-level water; and earthen dams with masonry core walls are too expensive.

II. Theoretical Considerations.

59. The Deficiency of the Theory of Earthwork.—Properly constructed dams are amply sufficient to resist the pressure of the water they hold up. The only way it can and does act prejudicially against them is by infiltration, thus diminishing their frictional resistance and adhesion. The risk of failure lies in the liability of the earthwork itself to slip. There have been many mathematical investigations as to the behaviour of earthwork, but, naturally, these have been confined to laboratory experiments; and, although they are most useful in indicating the character of the forces at work, they cannot, from the nature of things, be based on actual and comprehensive data, and cannot, therefore, give the actual amounts of those forces in all the varying circumstances which occur in practice. Sir Benjamin Baker,[1] Past President, Inst. C.E., has given numerous examples showing that the lateral pressure of earthwork against walls is, at most, only one-half of that pointed out by theory, and he states that practical considerations, rather than theoretical ones, should be taken into account when designing them to resist earth pressure. Mr. G. H. Darwin,[2] M.A., F.R.S., concludes: "The soundest view seems to be that engineers have no better practical course open to them than neglecting the elaborate formulas which have been suggested, to work with semi-empirical rules such as those of Coulomb, and to allow a large coefficient of safety."

Rankine[3] has stated: "There is a mathematical theory of the combined action of friction and adhesion in earth; but for want of experimental data its practical utility is doubtful."

[1] "Minutes of Proceedings Inst. C.E.," Vol. lxv., pp. 207 and 208.
[2] *Ibid.*, Vol. lxxi., p. 378.
[3] "Civil Engineering," p. 324.

60. General Causes affecting the Stability of Earthwork.—Earthwork gives way by the slipping, or sliding, of its parts on each other. The resistance to this is due, partly to the friction between the particles, and partly to their mutual adhesion, or cohesion.

The friction is measured by the angle of repose, and constants for it for different soils have been determined; these are co-efficients of the weight of the mass. It is greatest for coarse and least for fine soils; on it depends the permanent stability of natural earthwork.

The adhesion, or cohesion, may be measured by the depth to which an unsupported face of earthwork will stand. It is an extremely varying force, depending largely upon the condition of the material. It is increased by a moderate amount of moisture, but is diminished by excessive wetness. The cohesion of clays is considerable, and is largely increased by a small amount of moisture: a slight addition of moisture also increases the co-efficient of friction, but an excess of it acts as an unguent in diminishing it.

It is, therefore, evident that any given earthwork, other things being equal, will be most stable when slightly damp, and least stable when charged with water. Hence its stability depends upon the ease and thoroughness with which it can be drained of superfluous and dangerous water. Professor Rankine[1] sums up the matter thus: "The properties of earth with respect to adhesion and friction are so variable that the engineer should never trust to tables or to information obtained from books to guide him in designing earthworks, when he has it in his own power to obtain the necessary data, either by observation of existing earthworks in the same stratum, or by experiment."

61. The "Historical Element" of Earthwork.—There is a further cause of variation in the behaviour of

[1] "Civil Engineering," p. 317.

soils, and that is what Professor Clerk Maxwell has called the "historical element," which term not only comprises the manner in which the mass was put together, but also includes the different causes at work which have subsequently modified its condition.

In respect to the effect of the original method of the formation of earthwork on its stability and behaviour, Mr. G. H. Darwin,[1] M.A., F.R.S., has made experiments on "The Horizontal Thrust of a Mass of Sand," which show that when sand is formed in different ways into an embankment it produces different amounts of thrusts, although the mass presents the same external appearance. This fact is recognised in engineering practice, for, when it is desired that a mass of earthwork should settle tightly against a retaining wall, its constituent layers are made to slope downwards towards the wall; the steeper the slope, the greater will be their pressure. On the other hand, if the wall is to be freed from earth pressure, the layers are made to slope downwards away from it. The same thing occurs in Nature. "The stability of sedimentary rocks in the side of a cutting is greater when the beds are horizontal, or dip away from the cutting, than when they dip towards it."[2]

62. The Effect on Earthwork of Causes acting subsequently to its Construction.—In respect to the modification of the behaviour of earthwork by the effect of causes acting on it subsequently to its construction, so much attention has not always been paid, although it is equally necessary to take it into account. In nearly all earthworks the practice is to treat the material as homogeneous from top to base, and to adopt an uniform slope throughout. The lower portions in a large dam must, however, be in a very different condition to that of the upper ones, as

[1] "Minutes of Proceedings Inst. C.E.," Vol. lxxi.
[2] Rankine's "Civil Engineering," p. 318.

they are much more highly compressed and are moister. Probably the enormous superincumbent weight causes some stratification of the lower parts and diminishes their cohesion, while the increased smoothness, due to the pressure, lessens their frictional resistance. The amount of increase of frictional stress, according to the depth below the surface, depends upon the viscosity of the earth enabling it to transmit pressure, and this pressure must vary from point to point on the cross-section of the dam. The increase in moisture at the base will diminish both the frictional resistance and the cohesion. The variation of the materials, their disposition and the methods of construction, introduce further elements of change, so that there are numerous, entirely hidden forces at work whose magnitude and resultant action can be determined only by the experience of the works themselves.

In Nature it is seen that hills, or even large masses of soil, have not an even slope, but one that varies from steepness at the top to flatness at the base (*e.g.*, Fujiyama, in Japan). Although this form is partly due to the effects of denudation, it is also partly due to the natural slopes assumed by the material of the hills. Slips of earthwork show, at first, a similar, but more pronounced, outline. It is known, moreover, that the limit of height of ordinary earthen dams is comparatively low. French engineers have placed it at 60 feet, and, although there are many instances of greater heights having been successfully accomplished, a considerable amount of care is necessary to ensure the safe construction of dams of a height greater than 50 feet. Low dams can be constructed with much steeper slopes than high ones; the water-faces of dams require a flatter slope than the rear ones. From these considerations it may be deduced that, in an originally homogeneous dam with plane slopes, the resistance to slipping decreases with

the height from the top, and that the proper section for it is one having the slopes continuously flattened towards the base. An "empirical section" based on these principles is shown in dotted lines on Plate 5, Fig. 2. Taking the whole cross-section into account, it will be seen from this that a very considerable flattening of the base slopes results in a comparatively small increase of the original area.

63. The Behaviour of Puddling Clay with Water.—Experiment[1] has shown that a good natural specimen of clay when dried lost 25 per cent. of its weight, and 10 per cent. of its bulk; it then became extremely compact, and, if not allowed to expand, offered great resistance to the passage of water. A dried specimen of this clay reduced to a fine powder absorbed about 75 per cent. of its weight of water, and allowed of free percolation. When this powdered clay was pressed into a tube, 8 feet long and 3 inches in diameter, it absorbed 35 per cent. of its weight of water, but there were no traces of filtration through the tube. The compressed particles of clay, in absorbing the water, expanded so as to become watertight; the greater the pressure of the water, the more satisfactory were the results.

On a large work it would not be economically practicable thus thoroughly to desiccate clay, nor feasible to consolidate the whole mass if dry. The experiments show, however, the advisability of using no more water in the construction of earthwork than is necessary to produce a compact substance by rolling. The further compression of the mass should be given by the superincumbent weight of the dam, which should be allowed to act for as long a time as possible before the reservoir commences to fill, so as to prevent filtration through the green material. In a wet state clay reaches its

[1] *The Builder*, Vol. li., p. 400.

extreme point of expansion, and, when then exposed to the action of water, filtration must take place between the separated particles. Clay is, moreover, so retentive of water, that, if once soaked, it will be long before it parts with the excess of moisture; hence the greatest care should be taken during construction to use the minimum amount of water.

64. The Rate of Filtration through Soils.—The Depth of the Puddle Trench.—The rate of filtration of a soil depends upon its porosity, which governs the frictional resistance to flow, and the slope and length of the filamentary channels along which the water may be considered to pass. It is evident, therefore, that the direct rate of infiltration in a homogeneous soil must decrease from the top to the bottom of the puddle trench. The best section for a puddle trench is thus a wedge, such as an open excavation would give. It is true that the uppermost infiltering filaments, when stopped by the puddle, will endeavour to get under it, but a depth will be eventually reached when the frictional resistance along the natural passages will be greater than that due to the transverse passage of the puddle trench, and it is when this occurs that the latter may be stopped without danger, as the filtration to it will be less than that through it. This depth requires to be determined in each case, but in fairly compact Indian soils 30 feet will be a fair limit. Mr. David Gravell[1] cites the opinion of Sir Robert Rawlinson, that 30 feet depth of puddle trench is sufficient if a thick bed of concrete is placed at the back with a well for collecting water and a pipe leading this off to the downstream side; this was done in the case of the Yarrow Dam.

[1] *The Engineer*, Vol. lxiii., p. 189.

III. The Design of Dam Embankments.

65. The Section of the Dam Embankment.—The proper section to be adopted for a dam embankment depends upon :—

1. The angles of repose of the soil of which it is formed, when dry and when saturated with water;
2. The nature of the foundation ;
3. The height to which it has to be raised.

The following table gives the general sections which may be adopted with safety and economy for all ordinarily good soils properly consolidated :—

1	2	3	4	5	6
Height of Dam above Ground Level.	Height of Top of Dam above H.F.L.	Top Width	Upstream Slope.	Down-stream Slope.	Width of Dam at H.F.L.
	Feet.	Feet.	Ratio of Horizontal Width to Vertical Height.		Feet.
1. 15 feet and under	4—5	6	2 to 1	1½ to 1	20—23½
2. 15 feet to 25 feet	5—6	6	2½ to 1	2 to 1	28½—33
3. 25 feet to 50 feet	6	8	3 to 1	2 to 1	38
4. 50 feet to 75 feet	6—7	10	3 to 1	2 to 1	40—45

Above 75 feet in height special precautions have to be taken : these will be described below (*vide* paragraph 129).

In any one particular dam it is advisable to preserve much the same section throughout so as to improve the appearance of the work as a whole, and, also, because reductions at the flanks do not effect much saving unless these are very long and moderately high. In a long dam three changes at most will suffice, different sections being adopted for :—

1. The Gorge Embankment ;
2. The high part of the Embankment ;
3. The low Flank Embankment.

When such changes of section are made, they should be

carried out in lengths of 100 or 200 feet, instead of uniformly throughout the length of the dam, as this will make the work of setting out easier.

66. The Height of the Dam.—The height of the dam depends upon :—

1. The amount of foundation clearance;
2. The ground levels;
3. The full-supply and high-flood levels;
4. The amount of "free board," or the height of the top of the dam above H.F.L.;
5. The importance of the particular section of the dam, considered with reference to that of other parts of the embankment;
6. The allowance for settlement.

(1) Is dealt with in paragraphs 111, 113, 114, and 115 below;

(2) depend upon the longitudinal section of the dam line;

(3) the full-supply level depends upon the amount of storage to be impounded, and the high-flood level, upon the discharging capacity of the waste-weir.

(4) and (5) are dealt with in paragraph 67, and (6) in paragraph 72 below.

French engineers consider that the maximum safe height for an earthen dam is 60 feet, but embankments have been constructed in the Bombay Presidency up to a height of 80 feet. In England there are several instances of dams having been raised to 80 feet,[1] some to 100 feet,[2] and one to 125 feet.[3]

67. The "Free Board" of the Dam.—The dam has to be raised a certain height above the high-flood level as a

[1] "Minutes of Proceedings Inst. C.E.," Vol. clxxxii., p. 250.
[2] *Ibid.*, p. 208.
[3] *Ibid.*, p. 205.

matter of safety in ordinary circumstances, and to provide for any accidental settlement which may occur. This height depends not only upon the size of the reservoir, but also upon the importance of the particular section of the dam considered with reference to other parts of the embankment. In the case of large reservoirs, as the importance of their safety is greater, the free board should be larger than it is in the case of small ones, and, for this reason, it is better to raise the dams for the former at least one foot higher than those for the latter. Similarly, where the dam is of an extreme height, as where it crosses the bed of the impounded stream, it should be raised at least a foot higher than at the lower flanks, so that the danger may be avoided of a breach occurring from any cause at the former place, before doing so at the latter. A breach at the low flank will usually cause a small and less damaging flood than that which would occur were the breach to happen at the gorge embankment, and the subsequent repair work at the former would be relatively much cheaper to effect and would be safer afterwards than it would be at the latter.

The free board does not depend directly upon the full-supply level of the reservoir, as the required margin for safety has to be given above its high-flood level.

68. The Top-width of the Dam.—The top-width of the dam should be varied with the size of the reservoir and with the height of the dam, so as to be in accord with the general scale of the work, and, principally, to allow of the top-level of the dam being restored should any extra settlement occur. As the increase of top-width affects only slightly the total area of the section of the dam, it is, as a rule, best to maintain the top-width the same throughout the embankment, and, if any reduction is contemplated, to confine it to the low ends of the flank embankment.

Where such changes of top-width are decided upon, they should be carried out in lengths of 100 or 200 feet, so as to facilitate setting out, and should not be continued uniformly throughout the whole length of the embankment.

69. The Side-slopes of the Dam.—These primarily depend upon the nature of the material of which the dam consists. It will be explained later on (*vide* paragraph 80 below) that it is advisable to make this material of the same description in all cases, so that the same side-slopes should, everywhere, meet the conditions in force. Moreover, the side-slopes given in the table in paragraph 65 above have been found in practice, in numerous examples of different kinds of earthwork, to be reliable, as they are sufficiently flat to resist slipping.

The upstream slope is made flatter than the downstream one as it usually consists of more clayey materials and is saturated by the water of the reservoir.

When the embankment is higher than 75 feet, it will be necessary either to change these slopes or to make a difference in the design of the dam.

As shown in the table in paragraph 65 above, the lower the dam, the steeper may be its side-slopes; it is thus permissible slightly to steepen the side-slopes near the top of a dam (*vide* Plate 5, Fig. 2).

70. The Crest of the Dam—the Crest Wall.—In the earlier Bombay examples both the side-slopes above high-flood level were reduced to 1½ to 1 for reasons of economy. That practice is now condemned, as it does not allow any margin for making up any excess settlement that may occur.

There is another way of finishing off the dam, and that is by a crest wall as illustrated in Plate 5, Fig. 1. This has a section with faces battering both on the upstream and downstream sides, so that it cannot separate from the

earthwork, and so that it has ample strength with which to resist its thrust. To prevent any settlement from occurring, such a wall should have a wide concrete foundation, and should not be built until the dam has obtained a practically final consolidation. With proper arrangements such consolidation can be always attained, although, of course, the delay involved by them is not a recommendation in favour of this form of construction. The toe of the wall should be protected by a strong apron, 2 feet thick, of close-fitting pitching to prevent it from being undermined.

The advantages of this crest wall over the ordinary slope are that it :—

1. Protects the dam up to its extreme top from wave-wash, the inroads of vermin, and the growth of thick vegetation ;

2. Acts as a wave breaker and prevents waves from being carried over the dam in severe storms;

3. Lightens the top of the dam and saves the construction of a large amount of earthwork which would be entailed by extending the ordinary dam slope to the top of the dam ;

4. Permits of the raising of the dam for a moderate height when settlement occurs without a great reduction of its top-width, the crest wall being then continued with its original batters ;

5. Gives the work a better finish ;

6. Effects, at ordinary rates, some economy, compared with the ordinary continuous 3 to 1 slope, when the dam is over 27 feet in height.

Crest walls have, so far as is known, not been constructed in India on dams of any height, although there are many examples of old native works of small height which have been formed with an upstream wall backed by embankment. In England crest walls have been built, but possibly to a

different section, and with fewer precautions having been taken. The objections there raised to them are :—

1. Waves[1] wash up the pitched slope, strike the wall, and, rebounding, undermine its base;

2. Waves[2] are apt to be carried over the wall in stormy weather and thus erode the earth backing.

These objections are against the experience of harbour practice in which vertical walls are now preferred to slopes, as they are found to lessen wave action. The replacement of the top-slope of the dam by a crest wall is, however, not a matter of great importance, and the general practice hitherto has been against it.

71. The Width of the Dam at High-flood Level and Full-supply Level.—The width of the dam at high-flood level of course depends upon the height of the top of the dam above it, (taking the ordinary free board and the allowance for extra settlement into account), the side-slopes and the top-width. The figures in the table in paragraph 65 above show that this width is a considerable one, and is unnecessarily large if resistance to water infiltration had alone to determine it, for, in that case, a width of 6 feet would be ample.

The width at full-supply level cannot be given in the table, as it is equal to the width at high-flood level *plus* the sum of the ratio of the slopes multiplied by the high-flood depth, which depends upon the discharging power of the waste-weir and will thus vary for different tanks. It, also, is far in excess of the width which is required only to resist infiltration, and, generally, the widths of each part of the section of a well-constructed dam are, similarly, greatly in excess of those necessary for this purpose; they are, in fact, determined upon wholly different considerations, viz.,

[1] "Minutes of Proceedings Inst. C.E.," Vol. cxxxii., p. 205.
[2] *Ibid.*, p. 208.

the height, the side-slopes, and the top-width required for the dam as a whole.

72. Allowance for Settlement.—No matter how well a dam has been consolidated during its construction, its enormous weight, which far exceeds any that can be artificially brought to bear upon it, tends still further to compress the earthwork. The result of this amount of further compression is known as settlement, and provision for it must be made, both in setting out the work and in estimating its quantity. The vertical amount of settlement of a well-consolidated dam should vary from $\frac{1}{30}$th to $\frac{1}{36}$th of its total height, measured from its cleared foundation to its designed top.

The moist air of the monsoon and the rainfall then have a great effect upon the settlement of a dam, and it is probable that during the course of the first monsoon an embankment, although not called upon to impound water will attain half its final amount of settlement. If then, owing to the height of the section, it has to be completed in more than one working season, it will suffice, when setting out the work for the second and subsequent years, to consider that the finished base has already attained half its final settlement and to adjust the setting out of the upper part accordingly.

The final settlement of the dam will be practically attained in a few months after the full depth of storage comes against it. The dam will continue to settle for a few years more, but only to a very small extent, and, five years after water is admitted against it, there should not be any sensible further settlement.

It will not be right to depend entirely upon this property of the self-consolidation of a dam and wholly to neglect artificial consolidation. The latter enables the whole mass to settle uniformly and gradually, whereas, if the earthwork during construction were simply deposited in place, a

much larger amount of settlement would rapidly occur as soon as the dam became wetted, and, as this could not be uniform, owing to the varying heights in the longitudinal and cross sections of the work, internal stresses would be set up. Moreover, were the embankment composed originally of loose material, the water of the reservoir would find its way into it for a considerable distance and would tend still further to produce unequal settlement and greater internal stresses.

The more a dam is consolidated artificially, the less will be its subsequent settlement and the freer will it be from internal stresses, as its consolidation will be more uniform. For this reason, also, it is desirable to construct a dam slowly, so as to let its power of self-consolidation come gradually into play.

73. The Sectional Area of a Dam Embankment.—The sectional area of a dam with uniform side-slopes is given by the formula :—

$$A = \left\{T + H\left(\frac{S_1 + S_2}{2}\right)\right\} H$$

where A = the area in square feet;
T = the top width in feet;
H = the total height in feet; and
S_1 and S_2 = the ratios of the horizontal widths of the side slopes to their vertical height.

It will be seen from this that in high dams the top-width is a comparatively small factor and that the areas of different sections practically vary as the squares of their heights multiplied by half the sum of the ratios of their side-slopes.

In Appendix 20 are given tables of sectional areas for all ordinary top-widths, side-slopes, and heights varying by 0·1 of a foot; further refinement in taking out the height is unnecessary. It must, of course, be remembered that H is the sum of the natural height, the

allowance for settlement (paragraph 72 above), and the allowance for foundation clearance (paragraph 113 below).

74. Sections Required at Particular Sites.

(*a*) *Gorge Embankments.*—Where the gorge, or river crossing, is very high, a particular treatment is necessary: this is described in paragraph 128 below, which deals with dams with drystone toes. Where it is of moderate height, the dam will, as explained in paragraphs 66, 68 and 69 above, require a greater height, greater top-width, and possibly flatter slopes than when it is of small height. Particular attention should be paid to the benching of the side-slopes of the ground, as they will otherwise tend to cause the embankment to slip off them. To prevent this the benching should be designed with base-slopes inclined downwards from the natural gorge-slopes, so that the earthwork of the dam will tend to move towards the flanks rather than towards the river crossing.

Where the slopes are precipitous, it is better to substitute a masonry dam for an embankment, to avoid the tendency to slipping of the earthwork at such a place. The former construction will have the added advantage that undersluices can be made in it which will aid the waste-weir by bringing early and safely into action the "flood-absorptive capacity" of the tank (*vide* Chapter III., paragraph 184).

(*b*) *Dams on Inferior Foundations.*—A really bad foundation should, of course, be avoided, as no treatment, short of removing all the defective material, will enable the dam to be constructed safely. There are, however, other foundations which are not good, but which can be utilised by special treatment. The principal of this class is deep black "cotton-soil," which so often exists at the sites of dams. Where this occurs, the side-slopes of the dam should be widened at the base, so as to distribute the weight over as large an area as possible, and, as far as safety permits,

should be steepened at the top of the section so as to reduce the total weight. Particular care should be paid to drainage, and, a short distance inside the downstream toe of the dam, should be a deep and wide trench filled with pervious, sound and stable material to prevent the movement of the subsoil. The base of the downstream toe should be founded on a layer of murumey, or shaly, material, say 5 feet thick (*vide* also paragraph 114).

75. Breaching Sections.—In paragraph 66 (5) above it has been explained that the more important parts of the dam should be raised higher than the less important ones, but it is desirable that even the latter should not be injured by abnormal circumstances. To guard against such damage "breaching sections" should be introduced wherever safely practicable. These sections should, as a rule, have a height about 1 foot less than that of the flank embankment, their top-width should be reduced to 6 feet, and their side-slopes steepened as much as is quite safe in ordinary conditions. They should be located preferably where there are natural saddles in the ridge line, so that any flood caused by their being breached may be confined by the rising ground on each side. Where such a saddle does not exist, an artificial one can be formed by excavating, downstream of the breaching section, a channel to lead the flood safely away from the main dam. The excavated spoil can be generally used for the construction of the dam without increased cost.

The best position for a breaching section is where the channel from it can be led into the waste-weir tail channel, as then the flood from it will do the minimum amount of damage. For this reason, when the waste-weir is situated at an independent saddle, its flank embankments can be made to serve as breaching sections (Plate 4, Fig. 2, and Chapter III., paragraph 162).

It is not absolutely necessary that the breaching section

should breach automatically; it will suffice if its section is reduced in area so that it can be quickly cut away when necessity arises, for, owing to the slowness with which a tank rises, there generally will be ample time for this action to be taken so as to save the main dam from being destroyed.

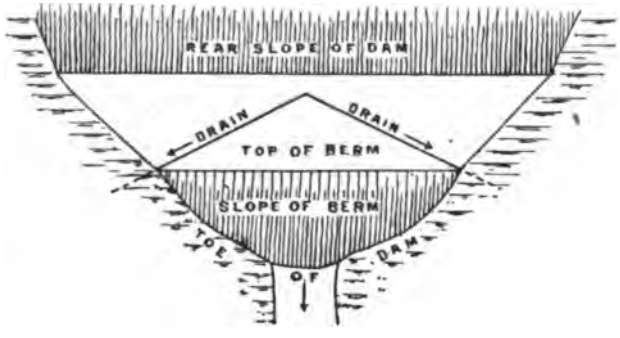
76. Berms.—In some large works, instead of flattening the downstream slope to get an increased base-width, the increase has been obtained by adding a berm, having the same, or nearly the same, side-slope as that of the upper part of the dam. Assuming the theory of the angle of rest of earthwork to be correct, the whole section is not in so stable a condition as it would be if the material of the berm were distributed throughout it so as to flatten its slope. The frictional resistance of a dam is a measure of its weight, and is independent of the area of its base, but its cohesive resistance is a measure of its base area; the construction of a berm increases that area more than the material in it would do were it distributed all down the slope, so that, in this respect, a berm is useful. The chief advantage of a berm is that, by its sudden increase of the section of the dam, it tends to prevent any dislocation at the top from extending to the base; it prevents any bulging of the subsoil: further, it also meets the case where the height of the dam is suddenly increased, as at the river crossing.

A berm is also useful for carrying a road across the river gorge and for passing off from the surface of the dam the drainage due to heavy rainfall, which amounts to a considerable quantity flowing at a high velocity when the dam is a high one, and, if not diverted from the base of the embankment, might cause the formation of scour channels. Fig. 2 shows how this can be done by means of paved, water-tight drains leading the drainage of the dam to the natural ground on its flanks. Such drains, if not water-tight, would be a source of danger, as they would tend to produce lines

of super-saturation in the heart of the earthwork which might lead to the formation of slips. It may be mentioned in passing that for this reason it is not advisable to form diagonal drains down the slope of the dam to pass off its drainage; in one case where these were laid, slips were thus caused.

Berms[1] were used in early English practice, but English engineers now appear to prefer gradually to flatten the slope of

FIG. 2.



PLAN

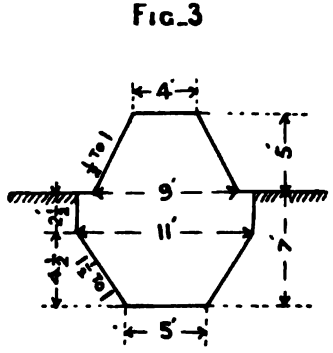
the dam than to step it out in a series of berms on which water might lodge. To prevent such lodgement of water, the top of a berm should have a slope of about 1 in 20 down stream.

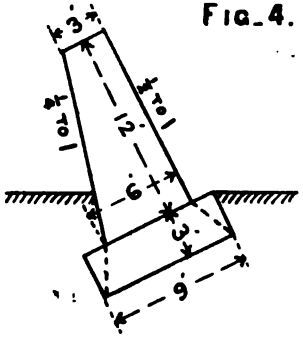
It is not recommended that there should be a series of berms up the slope of the dam, but a single berm at the base of a gorge will be often found useful.

77. Toe Walls.—A toe wall is another device with the same object in view as a berm at the base to prevent motion at the toe. Fig. 3 shows a form which is meant to secure the foundation of the dam and not its superstructure. A trench is excavated through the unreliable foundation and is filled with drystone which is weighted by the superstructure of the wall.

[1] "Minutes of Proceedings Inst. C.E.," Vol. cxxxii., pp. 219, 222, and 226.

Fig. 4 shows a form (really a retaining wall) which is meant to support the toe of the dam in the same way as does a berm. In all cases where earthwork has to be supported, it is best to make the section of the wall batter on both sides to the embankment, as shown, for it then offers an active resistance to the motion of the earthwork, and not merely the passive one of its stable weight. Upstream of the base of the wall a drystone drain should be laid to collect the internal drainage of the dam, and





this should be passed out of the wall by one or more slab drains, or large weep holes.

78. The Relative Cost of the Dam and the Waste-Weir.—The height of the dam depends upon the high-flood level of the reservoir, and this again upon the discharging capacity of the waste-weir and its crest level. The longer the waste-weir is made, the less will be the depth of the high-flood over it and the lower need be the dam. Comparative estimates of the dam of different heights and the weir of different lengths should be made in order to see which is the cheapest combination, remembering (*vide* Chapter III., paragraph 166) that the longer the weir, other things being equal, the safer the work.

Again, the level of the saddle, or ground in which the

weir is to be formed, may be higher, or lower, than what the full supply-storage capacity requires. It will be generally advisable to have the weir crest at the level which will most fully utilise the total yield from the catchment. When, however, the question of cutting down the ground extensively at the weir site arises, comparative trial estimates of the weir and the dam together should be made so as to see at what full-supply level the cheapest rate of storage can be obtained. The increase of the height of the dam does not appreciably affect the cost of the outlet, the buildings and the general preliminary charges, while the increase of its section will generally be less in proportion than the resulting increase in storage contents due to raising the full-supply level. It is generally better to have a storage capacity slightly too large than one which is too small for the catchment.

IV. Materials for Dam Construction.

79. Selection of Material Necessary.—The material of which a dam has to be formed requires careful selection: on the one hand, it has to be water-tight; on the other hand, while possessing adhesion, it should offer resistance to slipping. Powdery, dry material which will not bind, such as some kinds of marl (limy soil), and soils which turn into slush by the action of water, should be rejected. Pure sand has, however, been used for some dams; as it has the property of settling into a compact mass when wetted, and the angular form of the particles gives it a considerable amount of resistance to slipping. Sand in combination with clay is, however, not good, as it admits water into the latter material but does not allow it to drain out, while its particles, being so fine, do not give any amount of frictional stability to the clay in which they

are embedded. Pure black "cotton-soil" and rich clayey earths are dangerous, as, when wetted, they become greasy and treacherous, and are thus particularly liable to slips; they are, moreover, very retentive of water. The best material is one containing enough clayey matter to enable it to bind, and thus become water-tight, and enough gritty matter to give it frictional resistance to slipping and the property of self-drainage, so that the whole mass never becomes sodden.

80. The Disposition of Material in the Section.—In the earliest dams constructed in Bombay by British engineers there was a fear that the introduction of any material not itself water-tight would lead to infiltration of water, and thus be harmful to the stability of the work. Pure black "cotton-soil" was therefore used throughout the section (*vide* Fig. 5), and it was specified that all stony particles of any size were to be carefully removed.

The next step was to use this soil in the centre of the dam to form an impervious hearting; to weight it, and to prevent it from slipping, it was confined in place by side sections of "selected material" of heavy, shaly soil. On the upstream side this material was of an impervious nature, so as to resist the infiltration of water, and, on the downstream side, of a more shaly character, so as to resist any tendency to slipping. This section is sketched on page 78 (Fig. 5). It will be seen that it is virtually of the English type, the puddle wall of which is replaced by a much thicker one of clayey hearting of a less retentive nature than the puddle clay which is procurable in England, but which can seldom be had in India.

The final step taken, which is the one strongly recommended for adoption, was to use a soil of the same character uniformly throughout the dam, but, instead of depending solely upon its water-tightness, to select one

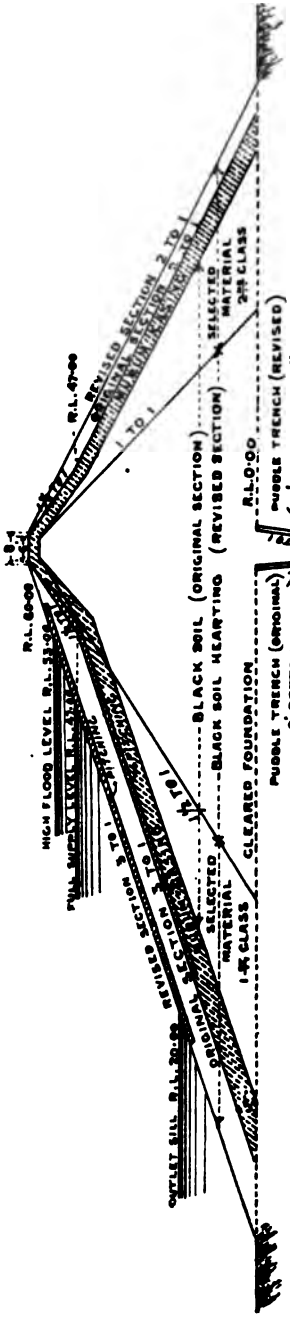


FIG. 5.

ORDINARY TYPE SECTIONS OF DAM 60.0' HIGH

that was both impervious and stable under the action of water. Such a soil is described at the end of paragraph 79 above. There are natural soils which answer to that description, but, where they do not exist within an economical distance from the site of a work—say half a mile—an artificial mixture should be made instead. The proportions recommended are 1 of pure black "cotton-soil," or other clayey soil, to 1 of pure murum, or shale. Where these materials are not found in a pure state, the existing soils should be mixed in the proportions which will result in the production of a similar mixture.

It must be remembered that, although the upstream slope is saturated by the water of the tank, it is adjusted to meet that saturation, and is under fairly settled conditions, for the water in the reservoir will not, as a rule, rise very rapidly during the monsoon, and it will fall very gradually owing to the draw-off during the fair weather. The downstream slope is much steeper, but is under less settled conditions, as it will become considerably and suddenly saturated during the monsoon downpours and less damp during the intervals between them, while in the fair weather it will be much dried. To meet this difference of conditions it is not unreasonable to form the downstream slope of material equally good and as carefully made as that of the upstream slope.

The great advantage of having the section of the dam homogeneous is that it will act uniformly as one mass during the process of settlement and self-consolidation, (which will last some years); moreover, the formation of internal stresses, owing to the different rates of settlement of its constituent parts if made of different materials, will be thus avoided. A difference of settlement might cause the formation of a slip, and will, anyhow, affect the disposition of the layers in which the dam was originally constructed.

There is also a practical consideration to be taken into account in regard to the method of construction. It is always easier to arrange for an uniform material throughout the section than for one varying in different parts.

81. The Casings of the Dam.—The casings of the dam may be considered to be entirely different from the main dam. Owing to their narrow width they will not affect it as a whole, and their object is a special one, namely, to protect the interior from external influences. On the upstream side it is necessary to form a firm and insoluble foundation for the pitching, and, on the downstream side, a covering which will not crack when dried by the sun nor gutter when subjected to rainfall. As the pressure of the reservoir water on the upstream side and the scouring action of rainfall on the downstream side increase directly as the height of the dam, the casings should be wider at its base than at its top. The widths, measured normally to the slopes, recommended are :—

At the top of the dam	2 feet.
From the top of the dam to H.F.L. on both sides	2 feet.
Below H.F.L. on both sides :—increasing uniformly at the rate of 1 foot in 10 feet vertical.	

The material of the casings should consist of a mixture equivalent to 1 of pure argillaceous soil to 2 of pure shale. The casings should be constructed uniformly with the hearting so as to be thoroughly bonded with and united to it.

82. The Utilisation of Spoil.—An economical advantage of mixing gritty material with the ordinary argillaceous earth is that all sound spoil from the excavations can be thus utilised in the formation of the dam. Where this grit

exists in large quantity beyond what is required to form the prescribed mixture, the excess can be incorporated with the material forming the downstream part of the dam, and the more stony portions can be reserved for the lower part of the downstream slope. The only spoil which should be rejected for incorporation in the dam is powdery, sandy or slushy material.

83. American Practice.—In American practice a still greater use of gritty material obtains. Mr. Fanning[1] recommends the following proportions for dams :—

	By volume.		Per cent.
Coarse gravel	1·00	cubic yard	59
Fine gravel	0·35	,,	20
Sand	0·15	,,	9
Clay	0·20	,,	12
Total when loose....	1·70	,,	100
Total when consolidated....	1·25	,,	

This mixture, when properly consolidated, would be free from voids, the proportions being adjusted for this purpose, but the small amount of clay used would apparently render the mass permeable, and would prevent it from possessing much cohesive stability. Mr. Clemens Herschel[2] distrusts the use of clayey material, and would not bring it on to the site of the works: he prefers a gravel that would puddle, or "binding gravel." To test the suitability of such a gravel for the construction of a dam, he would mix it with water in a pail to the consistency of moist earth as generally used in a dam. If on turning the pail upside down the gravel remained in the pail, it would be of the right character for use; but if it dropped out, it would be too gritty for employment, and should be rejected.

[1] "Hydraulic and Water Supply Engineering," 8th ed., p. 340.
[2] "Minutes of Proceedings Inst. C.E.," Vol. cxxxii., p. 253.

V. The Puddle Trench.

84. The Object of the Puddle Trench.—Practically no subsoils are water-tight, especially under the great pressure of water due to its storage in a reservoir. Ordinary earths may be of naturally porous material, or if of water-tight material, they may exist in layers between which water will find its way. Even when compact and clayey, the upstream particles, under the great pressure of the reservoir, will become charged with water, and will thus wet the ones downstream of them, so that gradually there will be a more or less slow passage of water through them. Rocky soils are very pervious if their particles are not cemented together to form a water-tight mass. Many rocks are stratified, and the spaces between the layers are either open or are filled by dry, porous material. Practically the only water-tight formation is one of dense, unfissured rock. The object of a puddle trench is to interpose a water-tight *septum* which will prevent the passage through it of the water coming to it, and thus will keep all the material downstream of it as dry as possible.

85. The Position of the Puddle Trench.—Taking only its water-intercepting object into account, the puddle trench should be placed as much upstream as practicable, so that the dry area downstream of it may be as large as possible. The limiting position in this respect is one where a sensible amount of infiltration through the superstructure of the dam will not pass over it and saturate the subsoil downstream of it. Another matter has, however, to be taken into account. The puddle trench filling, being of a different character to the natural strata through which the trench passes, and, usually, being originally of a more compressible nature, there will be at first unequal and generally greater settlement over it than on each side of it. For this

reason the position almost invariably selected for the puddle trench is on the centre line of the dam, so that the settlement of the superstructure may be uniform on each side of it. The greater weight of the superstructure along this line, where the height of the dam is at a maximum, also acts beneficially in compressing the material in the puddle trench to the greatest extent.

86. The Depth of the Puddle Trench.—It is necessary that the puddle trench should intercept such subsoil flow as would in the course of time carry away particles of the subsoil, thus enlarging the water passages and eventually causing the undermining and destruction of the dam. As it is not practicable after the completion of the embankment to make the puddle trench secure, it must be made quite safe at the time of its construction.

A less important consideration is that the puddle trench should prevent the leakage away of so much storage as would sensibly diminish the utility of the reservoir. With a trench of ordinary dimensions, this amount of loss is not likely to occur. It can be compensated for usually by a slight increase of the full storage level, seeing that the storage contents at the top contour of a reservoir are greater than those at the one immediately below it, and much greater than those at the lowest contours, while the increase of pressure, due to the small increase of level required, is practically inappreciable. In many cases also—such as where the canal or a minor channel is led off below the dam from the stream which supplies the reservoir—the water lost by leakage will be picked up by the weir forming the headworks of the distribution system.

The depth to which the puddle trench should be excavated depends principally upon the porosity of the soil through which it is carried, being made less for compact soils and greater for porous ones; it also depends greatly upon the head

of water in the reservoir which it has to resist. Assuming that good, compact soils are met with, the depth of the puddle trench need not ordinarily exceed one-half the depth of the full-supply storage; and that the soils are fairly compact, not more than the depth of the full-supply storage. The high-flood depth need not be considered in this connection, as the duration of floods will be ordinarily too short to increase the amount of subsoil flow to, or below, the puddle trench.

When the surface of sound, unfissured rock is near ground level, or not much below the depths above noted, the trench should be always carried at least 1 foot into it, as there will be nearly always a considerable amount of leakage along its junction with the top-lying soils. When, however, the rock lies at a much greater depth, the trench may be founded at the depths proposed above, provided it be carried at least 2 feet into good, compact and water-tight clayey soil, extending for some feet below its bed. If this foundation is not met with, the trench must be excavated deeper. It must be made to pass through all sandy and highly pervious layers, and, if these exist to a great depth, the idea of making a reservoir at the site may have to be abandoned.

Where the dam is situated on a narrow ridge of porous soil, or fissured rock, both the width and the depth of the trench should be increased to cut off the greater subsoil flow which may be expected there.

The nature of the subsoil is determined by sinking trial pits through it on the centre line of the dam. If these disclose a fairly regular disposition of the subsoil, they may be spaced up to 500 feet apart. If, however, that disposition is irregular, the trial pits should be nearer together, so that the strata may be correctly determined and plotted on the longitudinal section of the dam. The trial pits should be

invariably carried down to sound-rock level, so that its position and the nature of all the soils above it may be ascertained with a view to the formation of a decision as to how deep the puddle trench should be carried from point to point, and whether the foundation is reliable enough for carrying the dam.

Trial pits should be always excavated as near as possible to where minor drainages cross the dam line, as there the foundation may be less reliable, the strata may vary more rapidly, and the puddle trench may have to be carried lower than is necessary at adjacent places.

87. The Bottom-Width of the Puddle Trench.—The bottom-width of the puddle trench depends upon :—

(*a*) The nature of the strata near its foundation level;

(*b*) the nature of the filling material and the amount of consolidation to be given to the base of the puddle trench;

(*c*) the depth of the full-supply storage at the point and the importance and size of the reservoir.

Where the bottom strata are porous, the width must be made greater than it would be made were they compact. Where the total depth of the trench is small, the width must be increased beyond what is sufficient for a deep trench.

For large works it is necessary that the puddle trench filling should be of a more retentive quality and of greater consolidation, than are required in the case of small works: in the former case it is essential that the filling should be consolidated by rolling; in the latter, ramming will suffice for it.

Where the depth of the full-supply storage at the point is great, the storage is considerable, and the reservoir is important, the width must be made greater than it would be where the conditions require less care being taken to prevent leakage.

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As a general guide it may be said that the bottom-width of an important puddle trench should not be less than 10 feet, so as to give space for a roller to work, while for a small work it may be reduced to 6 feet. A smaller width than 6 feet is of little practical use in cutting off leakage. Subject to these restrictions, the base-width may be made equal to one quarter of the full-supply depth of the reservoir at the point considered; another rule for it is that it should not be less than one-eighth of the full-supply depth *plus* 3 feet.

88. The Side-Slopes of the Puddle Trench.—The side-slopes of the puddle trench have to be determined by two considerations :—

(*a*) They should be flat enough to stand during excavation and until the filling of the trench has been completed ;

(*b*) They should be flat enough to give the puddle filling a sufficiently increasing width as it rises towards ground level, so as to resist the greater infiltration it will have to withstand at the higher parts of its section.

In respect to (*a*) it may be noted that, as the time the trench will be open will be generally short, the slopes may be excavated to a steeper pitch than would be necessary were the excavation to remain permanently open. They should, however, not be excavated steeper than ¼ to 1 ; at this inclination the top-width of a trench 30 feet deep and with a base-width of 10 feet would be 25 feet.

There are certain formations where sandy pockets, or layers, occur irregularly. Where these are numerous it is advisable to widen the trench on the upstream side, 5 feet or more, and, as the filling rises, to pick out the sand from the pockets and layers, as far as they can be undermined with safety, and at once to fill them with puddle material.

89. The Length of the Puddle Trench.—At the full-supply margin of the tank the water pressure at the

surface is, of course, practically *nil*, and any considerable depth of trench there is generally unnecessary. Nor need the trench be extended longitudinally for any great distance, as floods usually will be short-lived, and therefore will not have time enough to develop much leakage. As the slope of the country will be generally flat at this level, it will, as a rule, suffice not to continue the trench beyond the high-flood contour. In England, in certain cases, the trench is made much longer, but that must be because the subsoil is more stratified there than it is usually in India. Of course, if a porous layer extends beyond the high-flood contour at a level below full-supply, it would lead to the out-flanking of the dam if not cut off by an extension of the puddle trench ; in such a case, the trench should be continued to prevent this.

90. The Height of the Puddle Filling.—The puddle material is usually carried up—say, 2 feet—above the natural surface of the ground so as to prevent the formation of a definite line of flow at this level across the base of the dam. This extra height should be constructed at the same time as the embankment on each side of it, and should be rolled and consolidated with it, the only difference between the parts being that of the materials of which they consist. If the completion of the puddle trench has to wait for the construction of the dam, the upper surface of the puddle should be kept a few inches below ground level for convenience of work until the embankment can be taken in hand. When the filling can be continued and completed, this temporary surface should be removed, until all cracked material has been taken out, and then the remainder of the puddle work should be finished and covered over with embankment for its protection.

91. The Continuity of the Puddle Trench.—If the puddle filling is disturbed so that a leak is formed through

it, its efficiency will be greatly impaired, and, as its material is of a naturally soluble character, it is possible that the leak may increase so as to become dangerous. The puddle being of a compressible nature, any sudden change in its section may cause unequal settlement and hence a leak. It is therefore essential that such sudden changes should be avoided, for which reason the bed should never be stepped up, but all changes of level should be effected by gentle slopes. Similarly, there should not be any projecting shoulders left in its side-slopes, as these may prevent the filling, during settlement, from occupying all the section below them; if this happens, spaces will be left in which the infiltering water may attain hydrostatic pressure sufficient to allow it to force itself as a definite stream through the puddle.

In short, the trench, both in longitudinal and in cross section, should be bounded by slopes, and thus be wedge-shaped, so that the only effect of settlement will be to make it more compact and to fill the excavation more completely.

92. Filling the Puddle Trench.—The bed of the trench should be first roughened, so that the filling may be thoroughly bonded with it and a definite line of flow between the two may be thus prevented; for this purpose the upstream half of the trench may be excavated as a wedge, say a foot deep below the downstream half. The natural surface of the bed should be cleared of all dry material, and, when clean, should be wetted to receive the filling. The bottom layer of the filling, about 1 foot thick, should be formed of carefully kneaded balls of clay thrown forcibly on to the bed and then thoroughly trodden so as perfectly to unite with it.

The subsequent filling should be carried out in layers, which, when completed, should not exceed 3 inches in thickness. These should be so laid that the whole mass

will be quite free from stratification and leakage planes; this can be avoided by wetting the surface of the completed layer just before the new one is formed on it, so that the material of the latter may be forced for a small depth into that of the former. As an additional precaution, at vertical and horizontal intervals, and breaking joint with each other, small wedge-shaped trenches, say 3 feet wide and 1 foot deep, should be excavated through the completed work, and should be refilled and consolidated just before the fresh layer is added. Further, to offer as much resistance as possible to the passage of water through the material, the layers should be tilted as steeply as practicable, say 1 in 6, rising from the upstream side.

The material of the puddle trench filling should be the most retentive clay procurable within half a mile of the site. A mixture with it of gritty stuff would make it more permeable, for, even if that material is itself water-tight, it and the clay will not be in perfect union, and water will tend to find its way between them and will have a shorter course from one stony particle to the next than it would have through a homogeneous mass of solid clay. In the superstructure of the dam a mass of pure clay is objectionable, as it may slip; but this it cannot do in the trench, for there it will be supported by the sides of the excavation, nor can it be forced out, being kept down by the weight of the embankment. In the dam itself a mixture of gritty material is useful in preventing shrinkage and the formation of cracks, in bonding the layers together, and in permitting the self-drainage of the earthwork so as to let it acquire greater stability. In the puddle trench it is not required for these purposes, as there the filling is under settled conditions. The filling should be constructed so quickly that no drying and cracking can take place, and the layers should be united together as above explained.

The mass should be therefore made as impermeable as possible.

The filling material should be deposited as dry as is consistent with its being thoroughly consolidated. As explained in paragraph 63 above, the wetting of clay causes it to expand, and any original moistening of it, beyond what is necessary to cause it to bind, will make it less dense and more permeable. The wetting that puddle (which was originally made with just a sufficiency of water) receives by percolation from the reservoir, subsequently to the completion of the dam, will cause its particles to swell and make the whole mass more compact and less permeable.

In the case of important trenches, the filling should be consolidated as much as possible by rollers. Further consolidation will result from the superincumbent weight of the dam, and this should be therefore allowed to act for as long as possible before the reservoir begins to fill, so as to prevent filtration through the green material.

Before the filling of the trench is commenced, all springs in it should be carefully led away in pipes, or otherwise, and, when the filling has reached a sufficient height, the pipes should be securely plugged.

In Appendix No. 19 are given tables for the calculation of the excavation and filling of puddle trenches.

93. Concrete Trenches.—The above paragraphs have dealt with earthen puddle trenches; but there is another class of an impervious *septum*—one made of concrete—which is much used in England, although it is not frequently employed in India on account of the expense of its construction. For concrete trenches the excavation is taken out in narrow timbered trenches, which in some English examples have been sunk as deep as 212 feet[1]

[1] "Minutes of Proceedings Inst. C.E.," Vol. cxxxii., p. 207.

below ground level. Such an extreme depth is only justifiable when the value of the impounded water is very great, *e.g.*, as in the case of a water-supply scheme, or where the lower strata are very porous and would otherwise constitute a danger to the dam. It is doubtful if even such a great depth would entirely prevent the passage of all water, although it would be useful in affording, on the down-stream side, a large extent of natural drainage area for passing off harmlessly the infiltering water which has got through the filling of the trench.

The thickness of a concrete trench depends partly upon the width necessary for its timbering, and partly upon the porosity of the soil passed through. The objection to a concrete trench is that the constituent layers are apt to make it stratified. To guard against this the surface of the completed layer should be roughened and should receive a thin coating of mortar before the new layer is formed on it. The concrete should not be thrown down from a height, as then the aggregate will separate and fall to the bottom of the layer and make it porous, but it should be carefully lowered and spread on the completed layer and then should be well rammed so as to fill the trench completely. The concrete should, moreover, be rich in mortar, the aggregate should be fine, and the layers should be laid in thin courses—say 4 inches thick—so that they may be consolidated uniformly throughout, one course being constructed at once on the top of the other, so as to get perfect bond with it. As many courses as practicable should be made at the same time to form a single layer and thus to reduce the number of horizontal leakage planes.

In India concrete would cost nearly eight times as much as puddle, and the excavation of the timbered trench, about as many times as the open one. Although the puddle usually obtainable in India is not so good as that procurable

in England, still the much greater thickness economically permissible with it renders it as safe to use as concrete.

94. Concrete Key Trenches.—Although a concrete trench is not generally used in India instead of a puddle trench, it may, with advantage, be employed at the base of the main trench to supplement it. If a hard pervious soil exists there, a narrow trench may be excavated in it and filled with fine concrete; the concrete should be carried up, say, 2 feet, above the bed of the main trench, so as to key into its puddle filling, care, of course, being taken properly to consolidate the projecting key (*vide* Fig. 8, paragraph 97).

If there is a layer of fissured rock at the bed of the puddle trench, it may be removed and be replaced by a layer of concrete across the whole bed, with a concrete key constructed on top of it.

95. The River Crossing of the Puddle Trench.—Not only is the river crossing the deepest part of the dam foundation, but it will probably have also the most fissured subsoil; extra precautions to cut off leakage should be therefore taken at it. These had best take the form of a thick wall, with masonry facings and a hearting of fine concrete, projecting some distance vertically into the body of the dam and continued laterally for some distance as sloping ramps into the flanks. There it should be keyed into the puddle trench, which should be widened out to overlap it on both the upstream and downstream sides. The foundation of the wall should be carried down into a perfectly sound, water-tight stratum, and preferably into rock (*vide* Plate 5, Figs. 2, 5, 6, and 7).

In such cases where it is necessary to make a water-tight junction between masonry and earthwork, the face of the former should be free from projections, and may, indeed, be plastered over with mortar or luted with clay. Water

is said to have an abhorrence to a right angle, for which reason small right-angled staunch pilasters should be formed at intervals along the wall, and, intermediately between them, should be small key recesses, so that the masonry and the earthwork may be joggled and united together.

Immediately to the downstream side of this concrete wall and parallel to it should be a continuous drain formed in the usual way and leading to the main rear drain (paragraph 108 below).

Where minor drainages are crossed, the puddle trench should be widened and deepened, and, if necessary, a concrete key trench should be constructed in it.

96. The Drainage of the Puddle Trench.—Notwithstanding all precautions, some water is sure to pass through the puddle trench. Certain English engineers dispute this, and state that the puddle trenches of their works are absolutely staunch. Possibly leakage does occur through them, but is not directly apparent, as it may be carried away unperceived through porous lower strata, and may not appear on the surface for some distance from the sites of the reservoirs. Hence, when the subsoil is pervious, the drainage of the puddle trench is not so necessary as when that is fairly impervious. In Indian reservoirs there is no doubt that subsoil percolation does occur; the levels of wells in the neighbourhood of the tank are raised and clear springs issue at natural outcrops below the works. Compared to the great length of the dams and the great pressure of the storages, the amount of loss from this source is very small, but still it takes place. In India the objection to leakage is not to its amount, which can be compensated for by a small increase to the contents of the tank, but to its effect in soddening the area below the dam. In a well-constructed dam, owing to its great thickness, there should

not be any percolation through it, and the actual junction of the base with the natural ground can be also made staunch owing to the great length through which the leakage has to find its way.

The case is different with the puddle trench. Compared with a dam 60 feet high, having a base-width of, say, 310 feet, a puddle trench 30 feet deep may have a top-width of only 25 feet, or only one-twelfth that of the base of the embankment, and it is enclosed on both sides by material of a more or less porous nature. Its own material is certainly not twelve times as staunch as that of the superstructure of the dam, and it would therefore seem reasonable to attribute the leakage which is visible below the reservoir to percolation through the puddle trench.

The questions to be decided are :—

(1) Should this leakage be allowed to continue to sodden the area downstream of the dam ? or, should an attempt be made to lead it away harmlessly ? and

(2) Can it be drained away without inducing a still greater amount of percolation, which may endanger the stability of the work ?

It is believed that the answers to these questions are that :—

(1) The soddening of the area below the dam is at least undesirable, even if it is not distinctly dangerous, and endeavours should be made to prevent it ; and

(2) The puddle trench may be drained with safety and without inducing a greater flow through it.

97. The Construction of the Puddle Trench Drain.— The Figs. 6, 7, and 8, opposite, show three forms of puddle trench with drains adapted to meet the conditions described in them. In each case a drystone drain is shown with a vent of 6 inches height and 4 inches width at the head : in a long puddle trench the vent might be increased gradually

FIG_6.

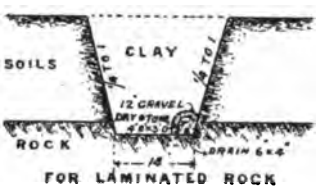


FIG_7.

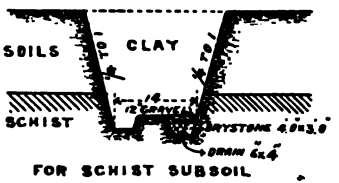
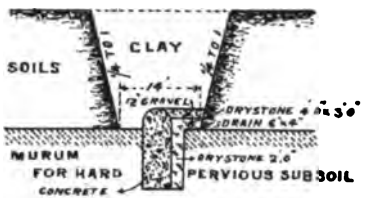


FIG_8.



PUDDLE TRENCH SECTIONS

to 9 inches square at the outfall of the drain. The minimum thickness of the sides of the drain should be 9 inches, and that of the slab covering, one varying from 6 inches for a 4 inch vent to 9 inches for a 9 inch vent. Surrounding the drain is a ring about 18 inches wide, of sound, clean and sharp small rubble; this, in its turn, is surrounded by a ring about 9 inches wide of clean gravel, small quarry spauls, &c., and this by a ring about 6 inches wide of clean, coarse sand. The object of these rings is to enclose the drain by filtering material (through which soil cannot be carried), so that the vent may be always clear.

The bed of the drain should be solidly founded to prevent any settlement, and should be formed of through slabs laid in mortar: the sides should be formed of long stones laid in mortar with narrow dry joints at intervals, of say 5 feet, to act as weep holes; the top should be of through slabs similarly laid in mortar, with narrow open joints at similar intervals breaking joint with those of the sides. The drain should slope continuously from its head to its tail, and no part should be lower than any portion of the length downstream of it. Where the puddle trench has a dip, the bed of the drain should be carried at its own regular inclination across it, being supported by a proper masonry foundation. The low part of the trench, at such a dip, can be drained upwardly by means of an iron pipe, having its outlet end carried up so as to discharge into the drain at its roof, and built in in masonry to secure a permanent vent (*vide* paragraph 108 below); or, better still, the whole of the low part of the bed of the puddle trench may be excavated to a narrow section and filled with concrete to the level of the bed of the drain.

As a rule, the clay filling of the base of the puddle trench should be first constructed for a height of about 1 foot above the top of the future sand cover of the drain, so as to

be consolidated thoroughly. The space to be occupied by the drain and its filtering cover should be then neatly excavated, the drain built and surrounded by the filtering material, and covered by at least 1 foot of clay carefully rammed; thereafter, the filling of the whole trench should be resumed for another 2 feet, giving, temporarily, a total earth cover of 3 feet above the filtering materials.

It is desirable to complete the whole length of the drain before the trench filling is raised any higher, so that its freedom from obstruction can be tested by passing water down it. Similarly, it may be advisable to finish off the upstream end of the drain by building it up along the downstream slope of the puddle trench, and then to carry it with a gentle upward slope out of the dam to a closed cistern, so that, if necessary, it may be tested and flushed with water at any time after the completion of the embankment.

98. Infiltration not Induced by the Puddle Trench Drain.—The construction of the drain necessitates the widening of the base of the puddle trench to a minimum width of, say, 14 feet, which, of course, entails so much extra expenditure, but is otherwise an advantage, as the wider trench will be all the more water-tight. As the drain will be formed at the downstream edge of the puddle trench, there will be a considerable thickness of puddle between it and the upstream edge, and, as the drain will be surrounded by fine material, there is not any reason why it should induce any excess amount of percolation. On the contrary, it should keep the superstructure of the puddle filling more compact and impervious. In a well-constructed puddle trench the subsoil water will tend to descend along the upstream face of the puddle to the base. If it does not there meet with a means of escape, it will endeavour to force itself up, thus rendering the filling less compact and more pervious. The drain, by leading away

this relatively small amount of percolation, provides this means of escape, and thus prevents the puddle filling from becoming deteriorated and the base of the dam from becoming sodden. In England, when springs are met with, they are generally led out of the trench by means of pipes, which is an arrangement for securing drainage at definite points. The above-described drain is designed to intercept percolation water throughout the whole length of the puddle trench and to pass it safely out of the dam.

It is, however, not advisable to continue the drain all the way up the downstream face of the puddle trench, for such a long, thin layer might suffer distortion or disturbance from the lateral pressure of the puddle or of the earthen sides of the trench, and might become discontinuous and thus harmful by collecting and impounding water. In the case of a concrete trench in hard soil, this would not occur, and the concrete with advantage might be backed by a rubble lining to act as a drain (*vide* Fig. 8).

99. Puddle Trenches not Suitable for Famine Work.—The excavation and filling of even ordinary puddle trenches have to be executed with comparative slowness, and should be carefully supervised; more especially will this be the case if base drains have to be formed. Such work is not, therefore, suitable for the employment of large bodies of famine relief labourers. Where the construction of storage works has been set aside for relief purposes, it is therefore advisable to complete the puddle trenches in advance. The filling should in such cases be carried up, say 2 feet, above ground level to protect it from being dried, and this 2 feet should be constructed of a fairly porous mixture to permit of the infiltration of rain water, which will keep the filling moist. When the dam has to be commenced, this 2 feet and any underlying part which

may be dried and cracked should be removed and refilled carefully with puddle material.

100. Nulla Puddle Trenches.—If the dam is on a sharply-defined ridge line, there may be at the base of the ridge a nulla, parallel to the dam and close to it, which may disclose porous strata below the level of the bed of the main puddle trench, and these would, if not dealt with, allow leakage water to pass below the trench. In such cases it is advisable to form a subsidiary puddle trench along the downstream edge, or side next the dam, of the nulla (*vide* Plate 4, Figs. 1 and 2, and Plate 5, Fig. 3).

After the reservoir is completed, its water will pond up along this nulla bed and retard the flow of the nulla water, so that fine silt will be rapidly deposited in the deeper part of the bed, *i.e.*, that nearest the main stream, and will fill it with fairly water-tight material.

101. Proposed Concrete Trench for the Bohio Dam, Panama Canal. — Mr. J. T. Ford,[1] M.Inst.C.E., has described a novel and ingenious arrangement and section for the construction of a concrete trench for an earthen dam across the large Chagres River, which is liable to bring down enormous floods. Such a trench may be 128 feet deep below the low-water surface of this perennially flowing river. Described shortly, he proposes (*vide* Figs. 9 and 10) to sink a shaft A on one bank beyond the reach of ordinary floods; to carry it down to bed rock; to build it above the reach of extraordinary floods; and to follow the bed rock on both sides from it by tunnelling, (freezing the water-bearing soil, if necessary), and using the shield form of excavation; or, if the conditions permit, by means of ordinary trench timbering. The surface of the sound rock having been cleansed, the tunnel is to be filled with concrete.

[1] *Engineering News*, Vol. xlviii., pp. 377–379, Nov. 6th, 1902.

PROPOSED BOHIO DAM — PANAMA CANAL

FIG. 9.

LONGITUDINAL SECTION.

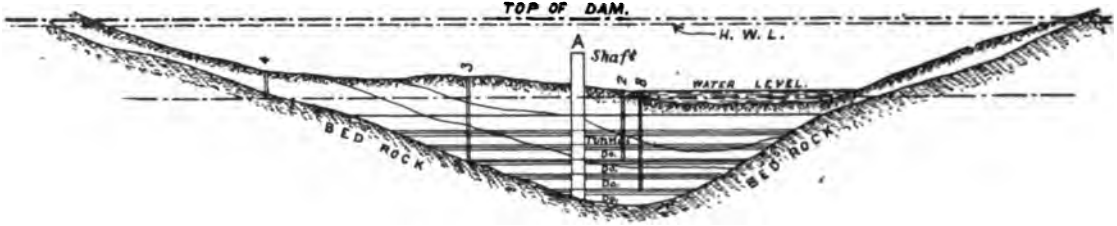
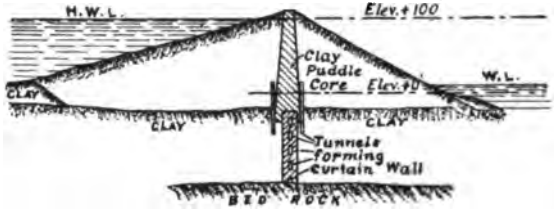


FIG. 10.

CROSS SECTION.



After the tunnel at the bottom has been thus filled, another one is to be excavated above it, and is to be filled similarly in perfectly water-tight connection with it. The trench is thus to be gradually completed by a series of similarly constructed layers in tunnels. The top section, replacing the bed of the river, might have to be constructed by open excavation with ordinary caisson and pneumatic work, sheet piling, cofferdams and dredging. He further proposes when filling the successive tunnels, to leave open culverts n them for the purpose of inspection at any time after the completion of the work.

The expense of such a construction would be so great as to prohibit its use in India, except, perhaps, in works of equal magnitude presenting similar difficulties. It is described as an ingenious proposal for meeting the conditions which exist at the site of the contemplated work.

He also proposes to make an ordinary puddle wall on top of the trench, but a masonry core wall w ould seem to be the better form of superstructure for such a foundation.

102. Excavation in the Reservoir Bed.—In order to prevent percolation under the dam as much as possible, it is advisable not to allow any excavation within the areas immediately next its toes. The minimum width of these areas, on the upstream and downstream sides respectively, might be four times and two times the height of the dam at the point considered. Further, the entire stripping off of the water-tight cover of pervious strata on the upstream side should be prohibited within a minimum width of ten times the height of the dam, measured from its upstream toe. Although such excavations will soon silt up, they will not become water-proof for many years, and, until they thus again become staunch, they may cause excess subsoil filtration, which may act prejudicially on the newly-formed dam.

VI. The Puddle Wall.

103. Central Puddle Walls.—In England it is the universal practice to have the puddle wall along the centre line of the dam and vertically over the puddle trench, so as to form with it a water-tight *septum*, (extending throughout the dam from below bed-rock level to above high-flood level), which is designed completely to intercept any infiltration which may have penetrated so far. The following are the sections[1] which have been adopted for the puddle wall for certain works :—

(*a*) Dale Dyke dam—top-width, 4 feet; batters, 1 in 16, giving a base-width of 16 feet where the dam is 96 feet high ; this section is very light.

(*b*) Vehar dam, Bombay waterworks (in this particular not a typical Indian dam)—top-width, 10 feet; batters 1 in 8.

(*c*) The Bann and Harelaw dams—top-width, 8 feet. Rankine[2] states that the thickness of the base of a puddle wall should be about one-third of its height, and that the thickness of the top should be two-thirds, or one-half, of that of the base.

The objects of placing the puddle wall in the centre of the dam are :—

(1) To make it in vertical continuation of the puddle trench ;

(2) To diminish its amount to the minimum ;

(3) To protect it from the action of the reservoir and the weather, and from damage by vermin.

The disadvantages of this position are :—

(*a*) It may be distorted or broken by unequal settlement of the embankment on each side of it, and during its own settlement, which will be considerable in amount ;

[1] *The Engineer*, Vol. lxiii., p. 189.
[2] Rankine's "Civil Engineering," p. 704.

(*b*) It is buried out of sight, and cannot be repaired for any considerable depth;

(*c*) The embankment upstream of it may be saturated to an undesirable extent.

104. Slope Puddle Walls.—Another position in which the puddle wall has been placed is on the upstream slope of the dam so as to form a water-proof covering to it. The advantages claimed for the puddle wall in this position are that:—

(1) It will settle regularly with the upstream slope;

(2) Being on the surface, it can be repaired at any time when necessary;

(3) It renders the whole mass of the dam as dry, and thus as stable, as possible.

The objections raised to it are that:—

(*a*) It has no direct connection with the puddle trench, and thus subsoil water is not intercepted at it and may enter and soak the dam;

(*b*) It involves the construction of a greatly increased amount of puddle work compared with that in a central wall;

(*c*) It is liable to be washed by waves in the reservoir, to become cracked when exposed to the sun, and to be penetrated by vermin;

(*d*) It may not be able to stand at the ordinary slope of the dam.

In some works the puddle wall has been placed between the upstream slope and the centre line of the dam. In such a position its advantages and disadvantages are between those of the ones described above. The general opinion of English engineers is much in favour of the central position for the puddle wall, but Indian and American[1] engineers, when they adopt a puddle wall, seem

[1] Schuyler's "Reservoirs," p. 281.

to prefer placing it on the external slope, in order to secure greater water-tightness of the dam as a whole.

105. Puddle Walls not now adopted in Indian Practice.—In modern Indian dams puddle walls have not usually been constructed on account of their disadvantages, as described above. Indian engineers prefer to construct the dam as one homogeneous whole and to make it solid and compact throughout, so that water cannot penetrate it to any great extent. The cheapness of Indian labour permits of this being done without great expense, but, as this compactness is obtained by rolling alone, it would not seem to be an extravagant precaution in any country thus thoroughly to consolidate the dam; moreover, it will secure the earthwork from the bad effects of an excessive amount of settlement. If, however, a water-tight *septum* is desired, it would seem better to adopt the masonry core wall (*vide* paragraph 58, *ante*) than the clay puddle wall, as it is less liable to failure, and because good puddling clay is not usually procurable in India.

The puddle wall is economical by limiting the amount of material to be made water-tight, but its use implies that the earthwork upstream of it is not reliable, and to have so large a mass saturated with water is most undesirable. The introduction of the puddle wall destroys the homogeneity of the dam, and this homogeneity is one of the principal objects which sound design and construction should secure.

VII. The Drainage of the Dam.

106. The Necessity for Drainage.—The proper drainage of the base of a dam is a matter of great importance, as, if arrangements for it are not made, it is possible that the embankment may subside, or slip, and instances of

such failure have occurred. It is easy during the construction of the work to take such precautions as are necessary, but very difficult to effect perfectly reliable repairs after it has been completed. Drainage should be carried out before damage occurs rather than undertaken after it has happened.

The maximum amount of percolation tends to occur along the junction of the dam with the ground, being derived, either from leakage from the reservoir finding its way along this course or up through the subsoil, or from moisture descending from the heart of the dam itself till it is stopped by the foundation. The natural surface of the ground, being weathered and usually not having been subjected to pressure, is less compact than are the subsoils of similar constitution. The action of the sun in drying up the surface tends to produce cracks, which, in the case of "cotton-soil," are often visible for several feet in depth, and may extend still deeper. With other soils these surface cracks are not so deep, but they probably exist in all varieties to a more or less extent. The proper treatment of the actual surface is described in paragraph 113 below.

107. The System of Drains Proposed.—The system of drains proposed is illustrated in Plate 4, Figs. 1 and 2, and Plate 5, Figs. 2 and 4, and is described below.

(*a*) *The "Surface Drain."*—There is no harm in dressing the surface of the ground at the rear of the down-stream toe to a small depth provided this excavation is drained and the formation of swamps is prevented. The general line of thrust due to the weight of the dam is inclined downwards, and the natural surface does not resist it as a buttress; it, of course, weights the subsoil and prevents it from rising, but the removal of a small amount of this weight is immaterial in this respect. It is far more important to prevent the whole of the subsoil at the rear of

the embankment from becoming sodden, for, if it becomes thus wet, it will tend to give under the weight of the dam. For this reason it will be best to slope off the ground just below the dam for a width of, say, 30 feet at an inclination of about 1 in 10 and with a longitudinal fall sufficient at once to carry away any rainfall running off the slope of the dam, even should weeds grow on the dressed surface.

This "surface drain" should be constructed in sections corresponding with the changes of the slope of the ground level and not more than 300 feet long, each section being separated from the neighbouring one by an unexcavated strip, say 10 feet wide, which will prevent the formation of longitudinal scour channels.

The flow at the downstream ends of these sections should be diverted from the dam by small outfall gutters (which should be carried in water-tight channels over the "downstream drain"), and should be discharged some distance below the embankment.

(*b*) *The "Downstream Drain."*—Just downstream and clear of this "surface drain" should be a parallel "downstream drain," which may have a base-width of 5 feet, side-slopes as steep as practicable up to ¼ to 1, and a depth of from 10 feet to 15 feet; it should preferably be carried down to 1 foot below the surface of unfissured, sound rock, if this exists within a moderate depth. It should have a small drystone base drain with a vent, say 4 inches wide and 6 inches deep, at the head, and, if the ground levels permit, should be divided into sections, separated from each other by unexcavated soil 10 feet wide, each leading out to the natural surface by outfall drains similarly constructed. If the ground levels do not allow of this being done, the drain should be continued until it can discharge into the river bed, the size of the vent being

gradually increased to pass off the drainage as it increases in amount.

The trench excavated for the drain, (which should, of course, have a continuous bed fall), should be filled up to 3 feet below ground level with quarry spauls and dry material from the waste-weir and other excavations. This filling should be of coarser particles at the base and of finer ones at the top, and should be lined at the sides and covered at the top by 1 foot of coarse sand and fine gravel. The whole should be finished off by 2 feet of soil, which, with advantage, may be carried a little above ground level, so as to mark the position of the trench and so as to prevent water from lodging above it.

This drain will effectually drain the ground on the downstream side of the dam and for some distance below it; it will also intercept and pass off any percolation arising from the puddle trench and prevent it from soddening the base of the dam. Further, it will have a decided effect in reducing the salt efflorescence which damages agricultural land below some dams. This efflorescence is due to the upward passage of subsoil water, charged with dissolved salts, in excess of that which the natural subsoil drainage can pass off. As this upward moisture is continually evaporated by the sun, the salts thus crystallise out on the surface and form an efflorescence, which is frequently fatal to plant growth.

The "foundation drains" (*vide* paragraph 112) below the downstream part of the dam can be led by cross drains at intervals into this "downstream drain."

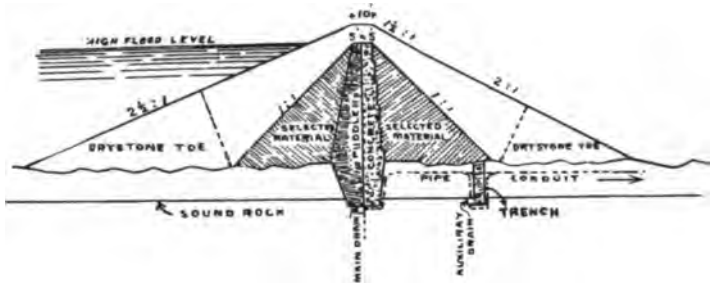
Mr. Kreuter[1] has proposed an "auxiliary drain," about half-way between the puddle trench and the downstream toe of the dam, with an auxiliary puddle trench

[1] "Minutes of Proceedings Inst. C.E.," Vol. cxxxii., p. 263.

downstream of it to prevent water from passing beyond it (*vide* Fig. 11).

He rightly observes that, not only is it necessary to prevent slipping from occurring within the mass of the dam, but also in the subsoils below it. It is to cut off percolation water from lubricating the subsoils, especially at their junction with the bed rock, that he has proposed this drain in addition to a puddle trench drain. The proposal is based on sound ideas, but, as the number of drains below the superstructure of the dam should be as few

FIG. 11.



as possible, it is questionable if this additional interior drain is necessary. The puddle trench drain and the downstream drain between them should keep the subsoil well drained.

He also proposes to make the core wall of two portions to secure perfect water-tightness—on the upstream side of plastic puddle and on the downstream side of rigid concrete. This seems to be a refinement which could not easily be carried out in practice.

108. "Rear Drains."—At all valley lines crossed by the dam, "rear drains" (Plate 4, Fig. 1, and Plate 5, Fig. 2) should be run at right angles to the centre line of the dam with as steep a bed slope as possible out to the surface of

the ground. They will pass off the flow from the "down-stream drain" and "puddle trench drain" to the natural drainage lines of the country. Where they are excavated in soil, they should be of the underground type (*vide* paragraph 109).

The "main rear drain" in the river bed will be generally in rock, and should, anyhow, be in open excavation, so that its free discharge can be observed and maintained at all times. It should be protected from cross drainage, bringing in silt and *débris*, by means of good side embankments, and care should be taken that its outfall is not choked by material brought down by the erosion of the waste-weir channel. As it has to drain the highest part of the dam, the greatest care is necessary always to keep it clear.

The bed of the puddle trench at the river crossing will be probably near the surface, and, in such cases, it can be drained directly into the main rear drain. Where, however, it is at a low level and the natural fall of the rear drain is gentle, it can be still drained into it by a short drain led upwards with a rapid slope from its bed and made to discharge above the full-supply level of the main rear drain, so that there may not be any back flow down it from that drain. The sloping drain, by affording an easy exit to the puddle trench drainage, will prevent the base of the puddle from becoming sodden. This artifice of upwardly draining subsoil water has been already tried with success in agricultural practice in cases where a retentive soil on rising ground is underlain by an imprisoned, permeable, and water-bearing stratum. It assimilates in principle to that of the artesian well.

To assist in maintaining the main rear drain clear, it is advisable to construct near its head a flushing pool, in which its drainage can be gradually stored and then passed out

rapidly at intervals to scour out any silt that may have been deposited in the lower part of its course. As the drainage water will itself be clear, this pool should not silt up, but probably it will have to be kept clear of weeds.

A small gauging weir notch should be formed above this pool, so that a daily record of the drainage discharge may be obtained.

As long as the drains run clear, it shows that no damage to the embankment is happening. Should, however, the flow be discoloured, it is an unmistakable sign that a leak is in process of formation, and, the longer such a flow continues, the greater will be the danger. The source of such excess percolation should be at once found out and the defective part of the embankment should be cut out and refilled, even if this involves the running of the storage to waste. Fortunately, such cases are practically unknown in Indian dams, and need never occur if the construction is sound.

A simple and convincing test of the efficiency of the drainage arrangements will be afforded by the condition of the ground at the rear of the dam. If that is free from excess moisture, it is certain that the drains are working properly. Swampy places, on the other hand, are sure indications that the subsoil percolation has not been properly intercepted by the drains, and the defective lengths should at once receive attention.

109. Open and Underground Drains.—Shallow drains in clayey soils are of little use for draining the rear of the dam, as they do not tap the deep-seated percolation water. This finally rises between them to the surface, and, being retained by it, soddens it and produces a marsh, in which grow in profusion reeds and water plants, which add a further obstacle to drainage. Such drains are also not useful for draining porous soils, which, before becoming

charged with water, can themselves pass off most of the supersaturation, although they may become deteriorated by it. Deep drains, extending if possible to the bed rock, are therefore required in both cases. If these drains are left as open trenches, they are liable to slips, to becoming choked with silt brought into them from the surface, and to being blocked by the growth of weeds, so that in a few years, if not properly maintained, they are likely to lose most of their efficiency. The system of underground drains described in paragraph 107 (*b*) above is therefore recommended; such drains will continue to run clear, as their sides cannot fall in, surface silt cannot be brought into them, and weeds cannot grow in them. As they can be generally filled by refuse dry spoil, their cost beyond that of ordinary borrow pit excavation for the dam will not be very great, and is well worth being incurred for the advantages they secure.

110. The Self-drainage of Earthwork.—The ordinary Indian soils available for dam construction are not absolutely water-tight; in fact, it may be doubted if such earths exist anywhere. A very retentive soil will thus admit a certain amount of water into its mass, but, as the hydrostatic pressure of this infiltering water diminishes the further it penetrates, if the section of the embankment is sufficiently wide, there will not be any percolation at its downstream limit, but the mass will become more and more charged with water the nearer the source of supply is approached. Retentive soils become greasy when wetted, and the worst varieties tend to become slushy when sodden. It is, therefore, most desirable to prevent the heart of an embankment from becoming saturated with water, and for this reason measures should be adopted to make it able to drain itself of superfluous water, which would become dangerous should it lubricate the whole mass, or should it

collect from a large area and find a definite line of escape. Such measures consist in mixing with pure argillaceous soils a certain amount of shaly material, which gives the dam this power of self-drainage at the same time that it increases its frictional resistance to slipping. If the dam is made wholly of dry, gritty material, it will, of course, allow of the free passage of water through it, and such a soil should not therefore be used for its construction. There is, however, a mixture affording a happy mean between excessive retentiveness and excessive porosity, and one which, without inducing percolation at the upstream part of the dam, will permit the largest extent of the downstream part of the earthwork to be self-drained and thus maintained in the most stable condition. The mixture recommended, (*vide* paragraph 80), is one part of pure black "cotton-soil" to one part of clean murum, or such other proportions of existing soils as will result in the formation of a material with the same proportions of clay and grit.

Theoretically, the parts of the section of a dam should become gradually more and more shaly and less and less clayey the further they are removed from the source of supply of the infiltering water. In practice, such a gradual change of material is difficult to carry out, and an uniform mixture is therefore desirable. If, however, there is an excess of gritty material available, it is best to utilise it in an increased proportion on the downstream side of the centre line of the dam.

VIII. The Foundations of the Dam.

111. Order of Suitability of Natural Foundations.—The best foundation for a dam is one of compact, unfissured rock, provided it is level, or does not slope downstream, as then it would tend to cause a slip. A small downwards

inclination of the surface upstream is not so great a defect, owing to the flatter upstream slope of the dam and to its being supported by the water pressure. To prevent any slipping of the earthwork on a slightly sloping base, the rock surface should be roughened by shallow trenches excavated in it parallel to the axis of the dam, and parallel core walls should be built at intervals projecting from it into the embankment.

The next best foundation is less compact rock, the unsound parts of which upstream of the puddle trench would have to be removed, but those which are down-stream of it might be allowed to remain for the sake of the means of drainage which they afford. Following this is compact murum. These three classes of foundation have the great merit that they will not subside under the immense weight of the dam.

After these come, in the order named, mán (a hard clay soil), brown and red soils, and black "cotton-soil"; they all require special precautions for drainage, as they will yield under a heavy weight when sodden, and if they are tilted to any great extent, may slip. Some engineers prefer a foundation of clay, so that the dam may unite perfectly with it, and consider that a rock foundation is liable to cause foundation leaks, but these can be prevented by a series of concrete trenches, whereas with clay it may not be possible to prevent deterioration of the foundations.

All soils which are light and powdery and wanting in cohesion, and all which under the action of water become slippery or slushy, are quite unsuitable either for the foundation or for the construction of a dam. Soils which contain carbonate of soda and deliquescent salts are particularly dangerous, owing to their power of dissolving, when wetted, the earth which contains them. If such soils exist for a moderate depth below the seat of the dam,

they should be entirely excavated and thrown to spoil If, however, they are deep and the expense of their complete removal is too great to be faced, the site will have to be rejected.

The essentials of a good foundation are that it should be of a compact nature, which will not yield when wetted, nor settle nor slip under the weight of the dam. It is not necessary to have for earthen dams the absolutely rigid base required for a masonry dam, but, the nearer the foundation of an earthen dam approaches this degree of excellence, the better. The thorough drainage of the bed of an embankment is a matter of vital necessity. To ensure it, it is always desirable to select a site for the dam where the fall of the main drainage lines will facilitate the rapid passage off of all subsoil water. Fortunately, most dams are constructed on ridge lines, which afford the required means of natural drainage owing to their elevation above the rest of the country.

112. "Foundation Trenches and Drains."—As explained in paragraph 106 above, the maximum amount of percolation will probably occur along the junction of the dam with its foundation; it is therefore necessary to take greater precautions here—firstly, to intercept it as much as possible, and, secondly, to drain off harmlessly that which has not been stopped.

Plate 5, Fig. 4 shows how this may be done. On the upstream side of the dam is a series of small puddle trenches, parallel to the main central one, of which one or more may be made of greater width and depth than the others to aid in preventing subsoil flow along a definite plane. Like the central puddle trench, they should be filled with as retentive material as can be found within half a mile of the site.

On the downstream side of the embankment is a similar

series of trenches, but these are filled in with porous material to form the "foundation drains," which will drain, uniformly and thoroughly, the whole of the overlying part of the dam. The porous material should be protected from silt infiltration by means of a sand cover, and should itself be arranged so as to have its coarser particles at the bottom and its finer ones at the top; it can be procured from suitable excavation spoil of a sound nature. It is not necessary that slab drains should be built at the bottom of these drains, as each of them will have to deal with but very little drainage. They should be excavated in discontinuous sections, say 300 feet long, falling uniformly to their downstream ends, where they should be connected by cross drains of similar section which should be carried out of the dam and continued well below the "surface drain" in covered slab vents leading to the "downstream drain" which will form their outfall. Each section should be separated from its neighbour by an unexcavated strip, say, 5 feet wide, and with its end face slightly sloped to the lower section, so as to prevent the formation of long continuous drains under the dam. If it is considered desirable to maintain a constant watch over the flow of the cross drains, they may be made to discharge into the "surface drain," but the better arrangement for this will be to form small inspection chambers in them where they emerge from the dam.

It may be noted that the nearest of these "foundation drains" will be so far removed from the reservoir that they cannot possibly induce any leakage from it.

This system of drainage is far superior to one that is sometimes adopted in which there is only a series of drains at right angles to the centre line of the dam, corresponding to the cross drains described above. Such right-angled drains, occurring, as they will do, at intervals only, will drain

the whole base irregularly, and will thus tend to produce unequal settlement.

113. Benched Foundations of the Dam.—The whole seat of the dam outside the puddle trench should be stripped of unreliable soil and should be then excavated into a series of large furrows parallel to the centre line of the dam and having their troughs above the "foundation trenches and drains" (*vide* Plate 5, Fig. 4).

The object of these benches is to let the dam rest on a series of slightly inclined planes, which will tend to make its layers settle towards its centre line, and not away from it, and will thus increase the stability of the whole mass. The downstream benches have the further advantage of leading all the percolation water from the overlying part of the dam uniformly to the "foundation drains," and of preventing the subsoil water from rising into the embankment. As the material excavated from the benches should be of good quality, it should be suitable for the formation of the neighbouring sections of the dam.

At the steep sides of the river gorge the benches should be formed in a different way, namely, at right angles to the axis of the dam and with a continuous slope towards the natural flank. Here the object of the benching is to make the embankment settle tightly both on to its base and on to the sides of the gorge. To prevent the formation of direct leakage planes, unexcavated strips, say 5 feet wide, should be left on the upstream side at intervals of, say, 50 feet, so as to close the furrows of the benches. On the upstream side of these strips the small puddle trenches should be excavated parallel to them so as further to aid in cutting off leakage. On the downstream side of the centre line of the dam the "foundation drains" should be excavated in the troughs of the furrows, and each should be led out of the dam independently of the others.

This preparation of the surface, being slow work, is not very suitable for the employment of famine labour, but, being all in the open, can be undertaken by it if started when the numbers are few, or if certain selected gangs are drafted to it after the numbers have become large. It could be done only at considerable expense in anticipation of famine work becoming necessary, as in that case the whole of the embankment would have to be raised a few feet above it to protect it from injury.

114. Deep Black "Cotton-soil" Foundations.—Where these exist, it may be desirable to fill the downstream benches, and to make the embankment over them for a few feet in height, with double the ordinary proportion of gritty material, so as to give the superstructure a firm, insoluble bed over the whole area. This and the flattening of the slopes on both the upstream and downstream sides should sufficiently distribute the pressure of the dam on the subsoil (*vide* also paragraph 74).

115. Sandy River Bed Foundations.—Where the river bed is of deep, compact sand with some clayey matter in it, it may be occasionally more economical to leave this unexcavated on the downstream side of the puddle trench, preventing its motion by a strong masonry toe-wall. The sand will thus act as a natural drain to the base of the dam. The part of the bed upstream of the puddle trench should, of course, entirely be cleared of sand, and, as a further precaution against direct infiltration, the puddle trench itself should be widened considerably. The chief danger from not excavating the downstream part will be the possibly unequal settlement of the two portions of the base of the dam, but this can be practically obviated by raising the dam slowly, and by giving it a year in which to settle after completion before the reservoir is filled. Should the expense of these precautions approach the cost

of the entire replacement of the sand by embankment, it is obvious that it will be better to effect that replacement rather than to economise a little by retaining the natural sand in the downstream part of the river bed.

IX. The Construction of the Dam Superstructure.

116. General Formation.—The method of constructing the dam is:—first to wet the layer last completed; on the wet surface to spread the material of the new layer; and then to consolidate it. On the completion of the layer, the process is similarly repeated for the next one, and so on. These operations are described in detail below.

117. Watering.—The sole use of water is to unite the constituent layers of the dam into one solid, unfissured mass, and the quantity of it used should be restricted so as to be sufficient only to form a thin film of moistened material on top of the completed layer, into which the dry material of the layer under construction may be forced during the process of its consolidation. Any excess of water beyond this will make the layer too moist and will expand its constituent particles, thus rendering it less compact as a whole (*vide* paragraph 63 above). Should a great excess of water be used, it will form slush, which will be evidenced by the upper layer moving before the roller with a wavy motion and cracking. Such slushy parts should be at once cut out, the excavated material should be put on one side and allowed to dry before it is again used, and fresh material should be carefully consolidated in its place.

Where the soil used in construction is very dry, it is a good arrangement to wet overnight the borrow pits from which it is obtained and then it will be found on the following day suitably damp for excavation and use.

After a few feet of embankment have been constructed, it is advisable, by means of a hose, to pour water over the slopes for several days until they become so consolidated by the washing in of the fine particles of the soil that the water no longer penetrates, but at once runs off them. The slope will be then found to be extremely compact for a depth of 3 or 4 feet, and to have so hard a surface that it will not be guttered by rainfall on the downstream side, while, on the upstream one, it will form a sound bed for the pitching.

118. Mixing and Spreading.—The clayey material should be first evenly deposited on the finished layer and should be then covered evenly with the gritty material, the thicknesses of the two being regulated by experiment so as to produce the finished mixture with the proper proportions of the two constituents. All clods should be broken up by hoes or wooden hammers. The two materials should be then thoroughly incorporated together; this can be done best by hoeing them by hand, but, as this is expensive, they may be mixed by harrows with long obliquely projecting wooden teeth, or by light inverting steel ploughs.

The greatest care must be taken to produce an uniform mixture and to avoid stratification, which not only might lead to infiltration along the porous planes, but to slipping along the clayey planes, should such be formed by imperfect mixing. The gritty material is required to give the whole mass frictional stability, to unite the different layers completely and intimately with each other, and to permit of the whole obtaining the power of self-drainage. The clayey material is required to give the whole mass cohesive stability and to make it resistant to water infiltration. The mixture being thoroughly made, the whole mass of the dam will settle and act uniformly under all conditions, and unequal stresses will be thus avoided.

The casings will, as explained in paragraph 81 above, be formed of material richer in grit; being constructed at the same time and being consolidated with the hearting, they will be perfectly united with it.

119. Consolidation.—In order to render the whole of the earthwork perfectly uniform, it is essential that it should be consolidated equally throughout the section, so that, when the final consolidation due to the settlement of the earthwork takes place, it may be perfectly regular and the original disposition of the layers may be maintained unchanged, and so that the formation of leakage and slipping planes may be prevented during that settlement. Should such planes be produced, it will not be possible at once to remove them, as their existence will not be known until they cause signs of failure.

In order to avoid an excessive and sudden amount of settlement, which may not act uniformly throughout the mass, the embankment should not be raised more than 30 feet in height in any one season. During the subsequent monsoon, at least half of the total final settlement will have occurred, and the substructure will be thus compact enough to permit of the safe raising of the superstructure upon it.

Entire dependence must not, however, be placed upon this self-consolidation of the dam, for, if artificial consolidation is not originally effected, the settlement of the mass will be much greater and much less uniform, owing to the varying heights of the embankment in its longitudinal and cross sections, and internal stresses may be set up. Moreover, an originally loose structure will be at first subject to great infiltration from rain and from the water of the reservoir, which will keep it green for a long time, and this will tend to cause still further unequal settlement and less resistance to further infiltration.

After the layer under construction has been mixed, a light

roller, weighing, say a quarter of a ton per foot run, should be passed over it to produce an even surface, on which a heavier roller, say 4 feet in diameter and length, and weighing three-quarters of a ton per foot run, can work easily. For small dams this latter roller will suffice to give enough consolidation, but for large ones it should be followed by a steam roller weighing ten tons. A roller worked by animal power should consolidate in a day 20,000 square feet, and a steam roller, 50,000 square feet of surface. Rough tests of the sufficiency of the rolling are that the roller will not further compress it, and that loaded carts passing over the surface will leave but faint rut marks on it. All rollers should be provided with automatic scrapers to prevent them from lifting the earth in caked masses, and for this reason cast-iron, and not stone, rollers should be used.

In America grooved rollers have been used so as to concentrate their weight on the area of their projecting rings; this will, however, render the compression uneven, and the chief advantage of their use appears to be that they will knead the layers together and prevent stratification. For the edges of the earthwork it is advisable to have split rollers, fixed on a common axle, and projecting beyond the track of the animals drawing them. At the top of the dam, where the lessened width will not allow animals to work, the consolidation should be effected by light rollers running, say, a quarter of a ton to the foot, and worked by men. One of these can be subsequently used for the maintenance of the top of the dam.

In confined situations, such as the junctions of two lengths of the dam, the ordinary roller cannot work, and resort will have to be had to hand ramming. The men should work in unison and be spaced apart a distance equal to a little more than twice the width of the bottom of the rammers, in order that they may strike the embankment

with them alternately on one side and then on the other, so as completely to cover the area, and to knead the earth together as they progress slowly forward. As ramming does not compress earthwork as much as does rolling, the places which are rammed are ones where the settlement will be eventually greater than that of those which are rolled, and, at the junction of the two, there will be a plane of unequal settlement. To avoid this as much as possible, long, continuous layers of earthwork should be constructed at the same time, and the junctions should be made in steps of low height, breaking joint well with each other (*vide* Fig. 12, paragraph 123).

Surface cracks and irregularities of the slopes may possibly occur during the monsoon when the dam is new. The cracks should be completely filled with a mixture of grit and soil well worked in by means of chisel-pointed poles 8 or 10 feet long. The surface, at places of settlement, should be slightly excavated and then restored by adding fresh material, which should be carefully consolidated by rammers and hand beaters, so as immediately to shed the rainfall.

120. Thickness of Layers.—The thickness of the layers has to be regulated so that the maximum consolidation is attained at the base of the dam, where the water pressure will be the greatest, and so that less and less consolidation is effected as the work rises and the water pressure decreases. It would cost too much to consolidate the whole dam throughout to the maximum extent, and this amount of consolidation is not necessitated elsewhere than at the base by the conditions which have there to be met.

To obtain this gradually increasing consolidation the first 30 feet of the dam measured from the top should be constructed in layers 5 inches thick when rolled by ordinary rollers ; the next 30 feet in 4-inch layers ; and the remainder of the dam and the river crossing in 3-inch layers. Where

steam rollers are used, these thicknesses of the layers may be increased by 50 per cent. The loose layers will roll down to about two-thirds of their original thickness. Owing to the great pressure ofthe dam, the shrinkage of the natural earth excavated for its construction will be about 6 per cent., *i.e.*, about 106 cubic feet of excavation will be required for 100 cubic feet of embankment, and this should be allowed for either in the borrow pit measurements or in the excavation rate.

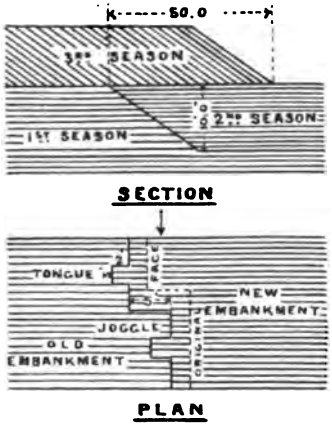
121. Slopes of Layers.—In order to counteract the natural tendency of earthwork to slip outwards, the layers should be formed with slopes inclined downwards to the centre line of the dam. The slope from the downstream edge should be made as steep as watering and consolidation will permit, say 1 in 10. The central eighth of the layer should be made level; the upstream edge should be kept at the same level as that of the downstream edge, and the slope from it adjusted accordingly. The resultant inwardly-slipping tendency of the downstream and upstream parts, having masses of different size and constructed with slopes of inversely proportioned inclination, will be thus approximately balanced. Care must be taken during construction that the water remains uniformly distributed over the surfaces of the inclined layers and does not run down them so as to collect at their bases.

During the self-consolidation of the dam, the interior portions being of greater vertical height than the exterior ones, and being subjected to greater pressure, even layers which were originally level must tend to slope down towards the centre, but to so small an extent that this natural settlement cannot by itself add appreciably to the frictional stability of the embankment.

122. Uniform Construction Necessary.—In order to avoid the formation of slipping planes, the whole width of

the section must be formed and consolidated at the same time. Moreover, as it is not possible thoroughly to consolidate the extreme edges, the width of the dam should be originally constructed 18 inches or 2 feet on each side in excess of the final width. After the dam has been raised a few feet, this extra width should be dressed off and the material from it used for making up the higher part. For the same reason no patchings-on of the casings, &c., should be allowed, and all works roads leading up the dam should be formed outside of its designed section and should be removed when no longer required.

FIG. 12.



Every precaution must be taken to construct the dam as carefully as possible so that it may settle uniformly under all conditions.

123. Junctions of Earthwork. — Where junctions of earthwork are unavoidable, they should be most carefully constructed.

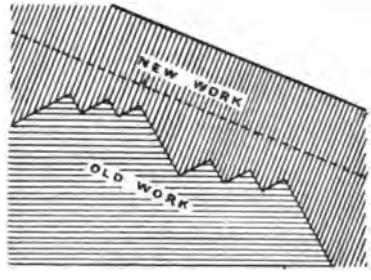
(*a*) *Cross-sectional Junctions.* —These should be made as shown in Fig. 12.

All the loose surface earth of the old slope should be entirely removed and the earth joggled, as shown in plan, with slip tongues inclined up the face of the slope to allow the new embankment to settle tightly on to the old one. Such junctions should not be made more than 10 feet in height, and, where a greater height has to be dealt with, it should be broken up into steps separated by horizontal breaks of at least 50 feet. The junctions should not be carried up more than 20 feet in height in one season, and

the work of the different seasons should break joint as shown in the sectional elevation.

(*b*) *Longitudinal Junctions.*—These junctions, or patchings, should be made as shown in Fig. 13, by benchings of irregular size, thus designed, so as to prevent the formation of a slipping plane; the new earthwork should, moreover, be constructed in layers sloping steeply on to the old surface. Transverse filtration will not be induced by such a junction, but, as this form of construction has somewhat of a tendency to slip, it should never be made if it is possible to avoid it. Not more than 20 feet vertically should be carried out in one season.

FIG. 13.



SECTION

Whenever new earthwork has to be added to the top of old earthwork, *e.g.*, when an old dam with a large top-width has to be raised, in addition to removing the old surface, one or more key trenches, say 4 feet wide by 3 feet deep, should be excavated in the old work with slightly sloping sides and parallel to its centre line. These should be filled with the most retentive material, thoroughly rammed, before the main part of the new earthwork is commenced.

124. Finishing-off the Dam.—At the end of each season's work on a section of the dam, the outer edges should be left a little higher than the centre, so that the rainfall on this wide top area may not flow down the slopes and gutter them, but, on the contrary, may soak gradually into the earthwork and help it to settle. Excess water should not be allowed to remain on any part of the top, but

should be drained off by gutters designed so as not to scour the slopes at their outfall.

The top foot of the dam should be made of porous material for the same reason, and the top surface should be given a slight fall, of say 1 inch, towards the reservoir, as the drainage from it will thus have only a short course to the pitching and will therefore not gutter the upstream slope. During construction care must be taken that the drainage descends evenly down the slopes, and that it is not allowed to concentrate down any definite line, as this may result in the formation of deep rain scores. The top of the dam should be finished off with half an inch of coarse sand, well rolled in.

To protect the downstream slope it should be sown with grass, or turfed, so as to enable it to resist the guttering action of the drainage of the rainfall passing down it.

A good fence or hedge should be made round the dam; on the upstream side this should extend down to full-supply level, and, on the downstream side, to outside the "downstream drain" (paragraph 107 (*b*) above), to protect the embankment from cattle, &c.

125. Observations during Construction.—To test the nature of the construction, at the close of each week small trial pits, say 2 feet square, should be excavated at intervals through the week's work. The earthwork, if it has been properly constructed, should be found only slightly moist, compact, uniform in composition, and free from distinct stratification. To test its impermeability, the pits may be filled with water and the time it takes to disappear may be noted. Any defects thus brought to notice should be remedied during the subsequent work.

During the rains theodolite and level observations of stout pegs driven 4 feet deep, projecting 1 foot above the surface and spaced at regular intervals on fixed lines ranged parallel to the centre line on all high parts of the dam,

should be made and continued until the close of the monsoon succeeding the completion of the work, and a continuous record of them should be maintained. Thereafter, if no great settlement or distortion has been observed, it will be sufficient to keep an annual tabulated record of the level of bench-mark stones firmly bedded 2 feet deep at the end of every chain on the top of the dam along its centre line; these stones should be engraved with the chainage for easy identification and so as to form permanent distance marks.

Gauging notches should be fixed at the outfalls of all drains and a continuous tabulated record should be kept of their discharges.

126. General Remarks.—(*a*) *Setting out the Centre Line.*—Dams should be set out in as long straight lines as possible, so that the appearance of the finished work may be in keeping with its scale. Following minutely the irregularities of the ground will not, as a rule, effect much saving compared with a carefully selected equalising line. These straight lines should be united by short curves, as long ones are difficult to set out, and there is no engineering necessity for them as exists in the case of a canal, road, or railway.

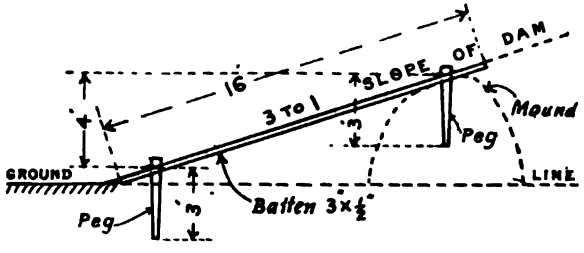
Reference setting-out pillars should be built at the extremities of all straight lines, and, if practicable, clear of the dam, so that they may be of use during the whole of its construction and also after its completion; from them the intersection points of the different lines can be always set out.

(*b*) *Setting out Templates.*—The toe of the dam should be set out as sketched below (Fig. 14), a small mound of the casing material being first formed to take the upper vertical peg. The template should be made by a good 3-inch by ½-inch sawn teak batten about 16 feet long,

as strings and bamboos are not sufficiently accurate for good work. These templates should be fixed 50 feet apart, opposite to and intermediate with, the regular chainage marks. After the embankment has been raised above the top of the template, the slope may be carried up roughly by eye, more especially since it has afterwards to be dressed off (*vide* paragraph 122 above). After it has been carried up above the length of a template, a second template can be fixed and the earthwork dressed off to it.

In the case of cuttings, excavation templates, about 1 foot wide, should be neatly dressed to the finished slope, at

FIG. 14.



intervals of 50 feet or so, and the earth lying between them should be dressed to them.

(*c*) *Minor Arrangements.*—In Appendix 22 are given numerous notes of these arrangements. These notes were originally issued as works orders, and will, it is hoped, prove useful.

(*d*) *Method of Executing Work.*—It will be generally found best to carry out a large dam by the petty contract system under departmental management, as a large contract is not, as a rule, advisable for this class of work, where the most careful construction is necessary, as well as rapid progress. Practically all the material being close at hand and easily worked, and nearly all the labour being

unskilled, there is not any necessity for the employment of a large contractor, whose principal use is that he has special appliances and trained labourers at his command.

X. Compound Dams with Drystone Toes.

127. Objections to High Earthen Dams.—As mentioned in paragraph 66 above, although French engineers consider 60 feet to be the safe maximum height for an embankment, English engineers do not agree with this opinion, as they have constructed many considerably higher dams. They generally consider that the proper section for high dams is one in which the slopes are made flatter towards the base than they are near the top. This corresponds with the "empirical section," paragraph 62 above, but that is not recommended as a perfect form for a high earthen dam. When a greater height than 60 feet is contemplated, it must be recognised that particular care must be taken both with the design and with the construction. Earthwork being viscous, a small disturbance at one point is likely to extend and to cause in time a larger one, and it is therefore necessary to guard against this. The higher the dam and the more that height differs at contiguous sections, the greater the chance of failure, as then there will be increased and varying settlement, and, unless this is provided against by careful and thorough consolidation, the more will it differ in amount at those sections, and internal stresses will be thus caused.

Now, the highest sections of dams are usually in the worst situations for stability. The most economical line for the whole dam is generally along ridges which terminate abruptly at the river, there forming a gorge through which the stream passes. At this point the steep side-slopes of the ground tend to make the embankment slip off, and, if

the bed of the gorge has a longitudinal slope falling downstream, which it frequently will have at such a site, this will accentuate the tendency to slipping. The upstream slope will be supported by the water pressure and by its greater mass, but the downstream slope will be dependent upon itself for support against the resultant force which will be directed towards it. The tendency will therefore be for the dam to slip downstream, and the slips which have occurred in a few Bombay dams have all been in this direction, no slips of the upstream portion being there known. Of course, if the upstream slope be steepened, so that, with the reduced co-efficients of cohesion and adhesion produced by water infiltration, it is relatively weaker than the downstream one, it will be the first to give. The result of experience is that the usual 3 to 1 water-slope, although it is charged with water, is more stable than the 2 to 1 downstream one, which is not exposed to the direct influence of the reservoir storage. In India this latter slope is, however, as described in paragraph 80 above, exposed to varying conditions which affect its stability; and, in the case of high dams, it appears necessary that there should be a less difference in its angle of slope to enable it to have as much slip-resisting power as the upstream slope. It is for this reason that the "empirical section" shown on Plate 5, Fig. 2, provides for a relatively greater increase beyond the usual section to the downstream than to the upstream slope.

128. The Drystone Toes of the "Compound Dam."—If the gorge embankment can have its sides confined by strong toes of some non-viscous material up to the base level of the flanks, (beyond which the dam is of a less abruptly varying height and where the difference of settlement of neighbouring sections will be relatively very small), the enclosed earthwork will be prevented from moving in

any direction. After it has been allowed to attain practically final consolidation by settlement, the upper part of the gorge dam can be safely raised on this reliable base with the ordinary section of the flank embankment. This constitutes the principle of what is here termed the "compound dam." The toes, if formed of trap drystone, which has a specific gravity of 2·50 and a coefficient of friction of 0·71, compared with 1·60 and 0·50, the relative figures for moist clay, will have a frictional resistance about 2¼ times that of the latter. To give them cohesion and to prevent on the upstream face the inflow of water, which would lessen the effective weight, the drystone should be packed tight with clayey murum. The stone would, of course, be the roughest of sound rubble, and probably much of it could be obtained from the waste-weir and other excavations. It should be laid with its beds normal to the slope to increase the frictional resistance.

129. The Construction of the Drystone Toe.—The toe could be constructed by unskilled labour in the following way:—On the completed layer of stone would be spread a thin layer of clayey murum thoroughly worked in to fill all its interstices. The stones in the next layer would be laid by hand to interlock and break joint with the underlying stones, and would be malleted firmly on to them. Large insterstices would be filled with single stones malleted home, and small ones with clayey murum thoroughly worked in by short bars, and then the layer would be completed by levelling it up with the murum so as to leave only the tops of the stones projecting in order to bond with the bases of those of the next layer. This construction should cost not more than three times the rate of the ordinary dam embankment. As all the material could be stacked beforehand close to the site, the drystone toe should be constructed at nearly the same pace as the

earthwork, which is a matter of great importance when the closure of the dam is taken into consideration.

To give greater frictional resistance and to prevent the infiltration of water, rubble masonry, or concrete, toes and core walls should be built into the foundation, so as to project into and key with the superstructure. The water face of the upstream toe should be laid in masonry, or covered with concrete, to prevent water from entering and buoying it up; its downstream face should be formed as a battering concrete wall to enable the earth pressure of the dam to be evenly distributed over it, and this, as explained in paragraph 136 below, will be useful in the closure of the gorge embankment.

It might be said that the upstream toe is not so necessary as the downstream one, owing, as noted above, to the greater stability of its flatter slope, but as this, after the filling of the reservoir, will be always under water and cannot be added to, it is advisable to construct it as a matter of precaution.

It might be also urged that the drystone toes might be replaced by masonry retaining walls, but the proper design for these with so great a surcharge would be somewhat doubtful, and sufficiently massive walls would probably exceed the proposed toes both in cost and in time for execution. It would certainly be hazardous to construct retaining walls of the same rough class of work, and the material saved by their use would not compensate for the loss of the extremely stable form of the proposed toe, which, in addition to its weight, receives the normal pressure of the dam and distributes it over a large, cohering base.

This type of dam with drystone toes is the one best adapted for embankments exceeding 75 feet in height, and with it dams might be safely constructed up to 125 feet in height.

130. The Advantages of the "Compound Dam."—In paragraph 53 above the advantages of earthen embankments have been described. In addition to those possessed by an ordinary earthen dam, the compound dam has the following:—

(1) It protects the base of the rear slope from being guttered by rain;

(2) It can be constructed to a considerably greater height;

(3) It can be raised subsequently by a largely increased amount;

(4) It offers great facilities for the safe closure of the work.

The compound dam is more expensive than the ordinary earthen dam, but is cheaper than a masonry dam or than a composite dam, which consists of separate sections of masonry and earthwork in continuation of each other. It has the advantage over a composite dam that its hearting is continuous and uniform throughout the whole embankment, and thus there is not any tendency to the formation of leakage planes through it.

XI. The Closure of the Dam.

131. The Importance of the Closure.—This portion of the work gives the Indian engineer the greatest anxiety, as a temporary passage for an immense quantity of flood water has to be provided until the beginning of the last season, when the last gap has to be closed within seven months. No considerable amount of work can be done after the commencement of the monsoon, as the reservoir is liable to be filled within a few days. During all the working period of the final closure a very large body of labourers must be kept constantly at work. The fear of an outbreak of

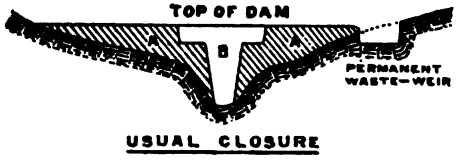
cholera, which would effectually stop all progress, is, also, ever present to him.

132. The Ordinary Method of Closure.—The ordinary system of the closure of a dam is shown in Fig. 15.

The flanks A A are first constructed, leaving the narrowest possible gap in the river bed for the passage of the monsoon discharge. The central part B has then to be completed during the last season of work. The objections to this method are:—

(*a*) The junction between A and B is too high. A, being made at least one season before B, and having been allowed to settle in the open, at the time of junction has attained its practically final consolidation, whereas B is quite green. The two cannot properly unite, and there is always a risk of a leak forming at their junction.

FIG. 15.



USUAL CLOSURE

(*b*) B, while quite green, is at once subjected to the infiltration due to a full reservoir; the water thus entering it prevents it from attaining the density of A for many years.

(*c*) B, having to be carried up the full height in one season, cannot be allowed to consolidate gradually by its own weight; it is, therefore, liable to internal stresses and distortions due to unequal settlement, which will be chiefly caused by the varying effect on the cross-section of the infiltration of water from the reservoir during the first few months.

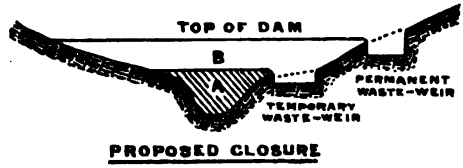
(*d*) B having to be completed in one season, the work is, to an undesirable extent, in the hands of the labourers, and strikes and higher rates are probable. Moreover, there is the fear of cholera interrupting its progress.

This method of closure can, however, be adopted without incurring much risk in the case of dams less than 30 feet high, but it does not seem advisable to employ it in that of ones of greater height.

133. The Revised Method of Closure.—The revised system of closure is shown in Fig. 16.

It consists in forming the dam in continuous level stages, commencing with the one at its lowest point, A, and working gradually up to that forming its final crest, B. To provide for the escape of flood water during the construction, a temporary waste-weir is made at a suitable site in the longitudinal section, so that it can discharge the maximum flood with safety to the whole work. This system of closure is the only safe one to adopt for dams of greater height than 30 feet. Its advantages are :—

FIG. 16.



PROPOSED CLOSURE.

(*a*) The junction between A and B is very much less in height than in the first case. The height can, moreover, if desirable, be reduced, at a small further expense by forming the top of A, near and along the junction, as a crest enclosure dam with reduced section; this can be removed, partly or wholly, when completing the dam at the time of final closure. In this case the top of the cross-section of the dam, as raised for the preliminary closure, would be stepped in profile, the crest dam being raised above the rest of the unfinished part of the main dam inside of it.

(*b*) A is subject only to infiltration from a low reservoir; all the upper part of the dam, with the exception of the comparatively low part closing the temporary

waste-weir, can be allowed to consolidate before water touches it.

(*c*) The bulk of the dam, and, in particular, its highest sections, can be carried up slowly and evenly and allowed to self-consolidate. Only the small portion closing the temporary waste-weir will have to be done in one season.

(*d*) The work can be executed without fear of strikes, as the final closure of B involves a comparatively small quantity of construction.

(*e*) The construction of A will lead to the formation of a small reservoir which will be most useful for works purposes, and will save much in water charges.

(*f*) The temporary waste-weir will often provide a level site for the location of the outlet, away from the gorge embankment and below the general bed of the dam.

134. The Temporary Waste-Weir.—As the temporary waste-weir is an essential to the closure of a high embankment, when selecting a site for such a dam it should always be seen that it offers facilities for the construction of this means of passing off floods safely during the progress of the work. In very high dams it may, of course, be necessary to extend the system by having one or more temporary waste-weirs at higher and higher levels, but the principle of execution will be the same as that to be followed in the case of a single one. The expense of forming more than one temporary waste-weir may be, of course, considerable, but, if an outlet of the type of the headwall in the centre line of the dam (Chapter IV., paragraph 207) is adopted and is located at the site of this weir, it will be generally possible to make use of only one temporary waste-weir during the entire construction of the dam. The method to be employed in this case is shown in Plate 4, Figs. 1 and 2. During the first season the temporary waste-weir is left as an

open channel. Afterwards, the lower part of the outlet headwall is raised to a certain height, calculated in accordance with the requirements of the construction of the dam, and its sluice-ways are left open to increase its discharging capacity.

The site for the permanent waste-weir should be kept open until the last (*i.e.*, the weir wall should not be built across it), so as to assist in discharging the flood waters at the lowest level.

The best natural site for a temporary waste-weir is at a depressed saddle, as then the floods from it will be confined in a natural channel, and will not tend to injure the main work. Where such a natural site does not exist, an artificial one should be made by excavating a channel so as to keep the tail flood below ground level. In either case it is advantageous that the bed of the temporary tail channel should be of rock, but, as this weir will come into action only for a year or two, it is not essential that the rock should be as sound as is desirable in the case of the permanent weir. Where sufficiently sound rock does not exist at the proper level for the bed of the temporary tail channel, the erosion of that bed, under and near the base of the dam, can be prevented by constructing a masonry curtain wall a little below the line of the toe of the dam, and also, if necessary, by building water-cushion walls downstream of this.

The discharging capacity of a temporary waste-weir must be equal to the maximum flood which can be expected from the catchment area, and, indeed, it will be better, as a precautionary measure, to make a somewhat greater provision than this, since, during the time this weir will be in operation, the whole of the works will be in an unfinished state. Account may be taken of the flood-absorptive capacity of the reservoir (Chapter III., paragraph 183), but

it must be remembered that this will be always small at the low level of the sill of the temporary waste-weir unless the maximum flood depth to be allowed over it is considerable.

135. The Closure of a Large Dam.—In a purely earthen dam the closure of the river gorge should be effected by carrying the full section right across the gap up to the required height above the temporary waste-weir. If, on the one hand, a reduced section is first constructed on the upstream side, and the downstream part is subsequently added, this latter will have a tendency to slip off during its settlement and under the immense pressure of this, the highest part of the dam. If, on the other hand, the width of the gorge is first reduced by patching on flank embankments, during settlement there will be a tendency to produce cracks and leakage planes at right angles to the section, and these will induce infiltration of water from the reservoir into the heart of the dam. By the system of forming the base of the gorge embankment with massive drystone toes, (paragraph 128 above), both these methods of reducing the amount of work involved by the closure may be adopted with perfect confidence, as these toes will give a large amount of support and prevent slipping, and their construction will prevent the formation of a direct leak through the dam.

As an example, the method of closure proposed for the large Máládevi dam is illustrated in Plate 4, Fig. 2, and Plate 5, Figs. 2 and 5, and may be thus described :—

136. The Preliminary Closure.—For this the whole section of the upstream drystone toe of the gorge embankment is to be raised to R.L. 124·00. A central gap (Plate 4, Fig. 2, and Plate 5, Figs. 2 and 5), 300 feet in crest-width, is to be left, and the flanks are to be sloped at an inclination of 1½ to 1 up to R.L. 166·00. The whole of the

downstream face is to be protected by the "concrete batter wall" from the action of the river floods. The earthwork of the dam hearting is to be set well back in low steps, out of reach of the direct rush of the floods, and its face is to be protected from erosion by heavy pitching, which will be subsequently removed and utilised for the permanent facing of the water-slope of the dam. The high-flood discharge is calculated to rise to about 20 feet on the sill of the gap, *i.e.*, to R.L. 144·00.

The raising of the flanks with full section to R.L. 166·00 provides an ample margin for safety, and this height might be reduced without risk if time did not allow for so much construction being carried out.

The "central wall" of the dam is to be fully completed with its overfall crest at R.L. 112·00. The width of its crest is to be contracted to 250 feet so that the flood water may be heaped up higher in order to secure a greater depth of water-cushion above it. The actual top of this wall, which is designed to key into the dam earthwork and which from its small section might be washed away by the floods, is left to be constructed afterwards during the first complete closure.

The flanks of the downstream toe with the concrete core walls are to be constructed so as to confine the river floods. Its concrete toe-wall is to be carried right across the river bed to form a water cushion. The central gap is to be widened to 325 feet in order to reduce the flood-level, as the toe-wall will give ample water-cushion depth for the small overfall at the "central wall."

137. The First Closure.—For this the temporary waste-weir channel at the outlet site has to be made ready (Plate 9, Figs. 1 and 3). It is to be constructed with a deep central channel, 50 feet wide, carried down to the outlet sill, R.L. 135·00, so as to tap the floods early and to bring

the reservoir flood-absorptive capacity (Chapter III., paragraph 184) sooner into play, and so as to form a channel to serve for the discharge of the permanent outlet. At each side is to be a side channel, 75 feet wide and with its bed 10 feet higher, which is to provide for the balance discharging power required.

Appendix 12 gives a calculation of the flood which can be disposed of by the combined effect of the channel discharge and the absorption in the reservoir during the rise of the water surface. Starting with a discharge of 5,300 cusecs (which is equal to a run-off of 1·28 inches a day from the catchment, and is a fair small flood), it will be seen that in nine hours it disposes of a total flood equivalent to a run-off of 4·75 inches. Of this, 2·99 inches are passed off by the channel, and the balance, 1·76 inches, is absorbed by the reservoir. When high-flood level is reached, and assuming the reservoir remains at this level, the discharge of the channel will be at the rate of 0·546 inch an hour run-off, thus giving a total run-off during the day of 12·94 inches. These runs-off are far in excess of what may be expected from the catchment, and the maximum high-flood level possible may be therefore safely taken as R.L. 160·00.

For this closure the upstream drystone toe of the gorge embankment has to be completed across the gap left for the preliminary closure, and the "minimum section for closure" (Plate 5, Fig. 2) of the earthwork of the dam has to be raised behind it. This reduced section is designed so that the quantity of work to be done may be practicable within the time available. Should progress be more rapid, as is desirable, then the downstream drystone toe and the rest of the earthwork between it and the "minimum section for closure" should be raised simultaneously. In any case it will be advisable to complete the whole section of the

base of the dam to R.L. 166·00 and allow it to settle for at least a whole monsoon before the upper part is raised upon it.

138. The Second Closure.—For this the outlet headwall has to be raised, (as shown in Plate 9, Fig. 1, by plain dotted lines), the under-sluice vents being left fully open in order to increase the discharging power. The upper plain dotted line indicates the high-flood level R.L. 166·00 as calculated in Appendix 13. As the temporary crest will be constructed in steps, for the sake of raising the superstructure safely upon it, it will give a larger high-flood discharge, and will tap the reservoir earlier than will the level crest at R.L. 155·00 which has been taken into account in the calculations.

The object of raising the reservoir flood-level above that of the first closure is to effect the raising of the dam gradually, so as to avoid leaving too much masonry and embankment work at the outlet for construction at the final closure. It will also ensure the more gradual wetting of the whole embankment and enable the part thus submerged to attain its practically final settlement before the dam is completed on top of it.

Appendix 13 gives a calculation of the flood which can be disposed of by the combined outlet headwall discharge and the reservoir absorption. Starting with a discharge of 5,706 cusecs (which is equal to a run-off of 1·39 inches per day from the catchment, and is a fair small flood), it will be seen that in eight hours it disposes of a total flood equivalent to a run-off of 3·98 inches. Of this, 2·18 inches are passed off by the headwall, and the balance, 1·80 inches, is absorbed by the reservoir. When high-flood level is reached, and assuming that the reservoir remains at this level, the discharge of the headwall will be at the rate of 0·42 inch per hour run-off, thus giving a total run-off

during the day of 10·69 inches. The maximum high-flood level may be therefore safely taken as R.L. 169·00.

For this closure the whole of the dam would be raised with full section at least to R.L. 175·00.

139. The Final Closure.—For this the under-sluice section of the waste-weir, 250 feet long, would be left open, the masonry foundations and superstructure being constructed to sill level R.L. 177·00, and the permanent tail channel excavated so as to form a temporary waste-weir channel. Detailed calculations of the flood of which it can dispose, aided by the discharge of the outlet sluices and by the absorption of the reservoir, are not given, but, judging from Appendices 12 and 13, the calculated high-flood level will not exceed R.L. 184·00, at which level the discharge from the permanent waste-weir and the outlet will be 20,624 cusecs, which is equal to a run-off of 0·21 inch per hour from the catchment. For the sake of safety the high-flood level may be taken as R.L. 187·00, when the total discharge will be 32,733 cusecs, or at the rate of 0·33 inch per hour run-off from the catchment. These discharges are exclusive of the large amount of flood-absorption by the reservoir.

If the rate of progress permits, the whole of the outlet headwall and the gap in the embankment at it (Plate 4, Fig. 2) should be completed in one season, as it is desirable to throw the temporary waste-weir out of action as soon as possible. If, however, the completion of these works cannot be then effected, these parts need not be raised at first beyond R.L. 190·00.

After all the works are completed, the reservoir should be kept at as low a maximum level for as long a time as possible, to permit of the practically final settlement and self-consolidation of the top of the embankment in the dry.

140. Quantities of Work to be done in each Closure. —The following table gives the estimated quantities of each class of work involved in each closure :—

1	2	3	4	5	6	7
Consecutive No.	Name of Closure.	Crest Reduced Level.	Earthwork. Cft.	Drystone. Cft.	Masonry and Concrete. Cft.	Total Work. Cft.
1	Preliminary Closure, Gorge embankment gap	124·00 / 166·00	2,673,000	1,772,000	491,000	4,936,000
2	First Closure, ditto, minimum section	166·00	2,494,000	740,000	130,000	3,364,000
3	First Closure, ditto, complete section	166·00	2,818,000	1,727,000	104,000	4,649,000
4	Second Closure	175·00	4,001,000	49,000	59,000	4,109,000
5	Final Closure, full height ...	207·00	565,000	9,000	290,000	864,000
6	Total Closure	207·00	12,551,000	4,297,000	1,074,000	17,922,000

In these figures no flank embankment work is included, as that can be completed previously to the actual closures concerned.

Owing to the limited extent of the working area and to the necessity for careful and slow construction, it is not safe to reckon upon a greater annual progress than 50 lakhs of cubic feet for each of the closures. For the closure in one operation of another design for an entirely earthen dam of about the same size for the Máládevi tank, it was estimated that 13,642,000 cubic feet of earthwork would have to be completed in one season, and, by reducing the gap beforehand by side embankments, that 11,036,000 cubic feet would thus have to be completed. It is very doubtful if even the smaller of these two quantities would have been feasible in the comparatively short working season of about 180 days, and the raising in it of so high a mass of earthwork would not have been conducive to its final stability.

Each of the above described closures will involve the construction of temporary work and its future replacement by permanent work, but the cost of this will not be great.

It is essential for all closures to make out a careful programme of the work to be done each month, and to keep the actual construction as much as possible in advance of it to provide for the delays which may occur later on.

XII. Earthwork Slips and their Repair.

141. The Necessity for the Sound Construction of the Dam.—If an ordinary high earthen dam is not properly constructed, it may be liable to sudden and unforeseen slips, which may lead to its failure, may cause consequent damage to life and property, and may entail loss of revenue and expensive measures of repair. It is far better to obviate all chance of failure at some extra capital outlay during the construction of the work than to run any chance of risk for the sake of a comparatively small original economy. Precautionary measures to be adopted in the design and during construction have been described above, and, where these have been taken, the chance of failure will practically disappear.

A slip is the only form of damage which need be considered here. The overtopping of a dam is due to insufficient waste-weir provision, and the failure of an outlet is due to mistakes which will be pointed out in Chapter IV., paragraph 201. A slip may be caused in the following ways, by—faulty foundations; unequal settlement, producing internal stress and subsequent motion; defects in the construction and design of the dam; changes in the chemical and physical constitution of the material of the dam or of the subsoil; and by infiltration of water.

142. Faulty Foundations.—Foundations may be faulty from two causes: they may be compressible or they may be badly seated. In regard to the first of these it may be

stated that most dry soils can withstand the weight of a dam. Argillaceous, or partially impervious ones, are, however, likely to give when saturated. A deep clay seat is therefore undesirable for a dam; and, where it exists, it must be rendered as compact and dry as possible on the downstream side, by a series of deep, dry rubble drains parallel to the centre line and with cross outfalls as frequently as can be arranged. On both sides the slopes of the dam should be flattened so as to secure a wider base, and, if necessary, the natural soil on the downstream side should be replaced by more reliable material. Berms, also, may be added at the bases of the slopes to prevent the rise of the subsoil.

If the dam is constructed on a stratum which is tilted and rests on a lower one with which it is not firmly united, the extra weight thus put upon the stratum may cause it to slide and carry the embankment with it. Careful geological investigation is necessary to avoid this source of danger.

143. Unequal Settlement.—Wherever the longitudinal section of the dam varies greatly in height, and wherever the construction has not been uniform and slow, or junctions have been formed, there is a risk of the occurrence of unequal settlement, which will cause internal stresses and subsequent motion. The effect would be intensified by the action of the water, which might thereby percolate from the reservoir, thus saturating the earthwork and forming a slipping plane. The place where this will most frequently occur is, of course, that where the dam was closed, should this have been done in the usual way at the river crossing (paragraph 132 above). Slips have occurred in some Bombay dams at this place, and years after their construction, showing that some slow disintegrating action, such as that described, has been at work.

144. Defects in Construction and Design.—The use of too permeable materials will cause a breach rather than a slip; but with ordinary care in construction, such materials become too compact to allow of dangerously excessive percolation. Friable materials, too loose to bind and totally wanting in cohesion, may form a slipping plane and lead to failure. Pure clays are dangerous in that their cohesive and frictional resistances become very largely reduced when they are charged with water, and a too liberal use of water during construction is, therefore, to be strongly deprecated. Through a defective design the earthwork may be unable to support its own weight, and will slip so as to assume more suitable slopes. The profile which is only just sufficient for a dry embankment will prove too slight when it is subjected to water infiltration.

145. Changes in Chemical Constitution.[1] — Some clayey materials are liable to changes in chemical constitution, leading either to deterioration or to disintegration. Occasionally, clay is found to be hard and tenacious originally and to maintain this character as long as it is under water, but, after having been allowed to dry and having been again placed under water, to turn into an impalpable mud, showing that some change has taken place in it; this is probably owing to some salt, such as carbonate of soda, contained in it. Soils which show an efflorescence on their surface should not therefore be used in the construction of a dam.

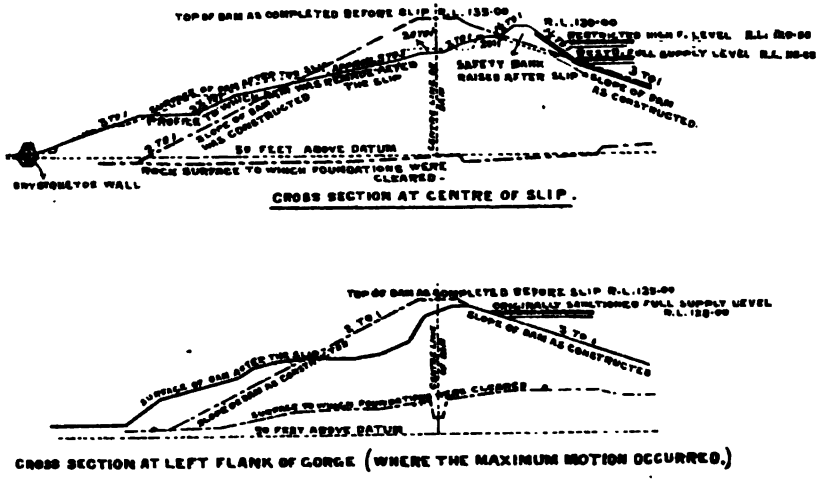
146. Infiltration of Water.—This is the principal cause of slips, as it operates in aiding all the others to produce failure. Thorough drainage of the earthwork and of its foundations is, therefore, essential to secure stability.

147. Profile of Slips.—Although Indian dams are

[1] "Minutes of Proceedings of the Inst. C.E.," Vol. cxxxii., p. 211. *Engineering News*. Vol. xlix., No. 2, dated Jan. 8th, 1903, p. 38.

carefully constructed, still slips sometimes occur in them. Frequently the initial cause is obscured by the fact that several causes are at work together. These slips, when extensive, are of the same general form—S-shaped in vertical section, with the concave portion at the top and the convex at the bottom, and, in plan, concave to the original toe of the dam (*vide*, Fig. 17). The layers at the base of a slip are generally tilted at an angle of about

FIG. 17



20° to the horizon, so that their inward thrust preserves equilibrium with the outward one of the dam.

148. Slips in Pure Black "Cotton-soil."—Where a slip occurs in pure black "cotton-soil," its surface presents a smooth, unctuous appearance, striated by the small particles of contained grit, and parallel planes of similar surface are formed for some feet on both sides of it. It is doubtful if these will ever disappear of themselves; the result is that the fallen mass rests on a series of

steeply-tilted, smooth, lubricated planes, and a small additional weight, produced either by adding more material to it, or by the soakage of rain-water, will tend to cause further motion. The whole of the slipped earth has also lost all the consolidation artificially given to it during construction. It is traversed by minor slipping planes and by fissures, which will admit rain-water and cause it to have a greatly reduced frictional and cohesive resistance to motion. No dependence can be therefore placed upon it, and the only sound system of repair consists in entirely removing it and in replacing it by trustworthy material.

149. Slips in Gritty Soil.—Where a slip occurs in earthwork having proper proportions of clay and grit, the latter will enable the whole to reunite gradually, but the junction will be, of course, always a plane of weakness. This property of reunion of gritty soils is a valuable one. The slipped portion will be also fissured like one of pure clay, but the gritty particles will admit of slow self-drainage, so that, after a couple of years, the earthwork will attain a fair amount of self-consolidation. The fallen mass when properly drained may be, therefore, utilised in reconstruction, care being taken to weight it very gradually, so as to let each season's work attain a practically final consolidation before the next is raised upon it, and to maintain instrumental observations to detect the smallest motion. When this occurs, further additional weight should not be put on until the toe is properly buttressed.

150. Repair of Slips—Preliminary Operations.—The first operation in all repairs is to drain the slipped earthwork by drains at right angles to the axis of the dam. Of these one should be at the centre of the disturbance, and two should be along the junctions of the slip with the solid embankment; others should be inserted at convenient, intermediate distances. They should be taken out in

timbered trenches, extending from the toe of the slip to a little beyond its innermost slipping plane, and should extend vertically throughout the mass. As soon as the excavation of each is completed, a longitudinal base drain with a vent 4 inches broad and 6 inches deep should be laid, and the trench should be filled with drystone, having gravel casings at the sides and a 2-foot cover of fine stuff and earth at the top; otherwise, it will rapidly choke. Not only do these drains serve their initial purpose of passing out soakage water harmlessly, thus allowing the fallen mass to consolidate itself, but they also divide it into independent sections. It is unlikely that all these sections will tend to slip at once, and thus each at the time of the initial tendency to motion is supported by the resistance of a length of the toe works considerably longer than itself. Where practicable, and when the fallen mass has been drained, the whole line of the slip should be followed up and cut out so as to get rid of the slip planes. This may be done in a difficult case by means of a timbered trench. The re-filling should consist of gritty clay, with a good base drain communicating with the cross drains above mentioned.

151. Repair of Slips—Final Operations.—The next step is to construct a strong, well-drained drystone wall parallel to the axis of the dam, and with good batters against the slip (*vide* Fig. 4, paragraph 77 above), so as to give the new earthwork the needful increased stability, for it will not have, bulk for bulk, the resistance of the original construction. If the fallen earthwork is to be entirely removed, this wall may be placed at about the centre of the width of the slip; if that is to be allowed to stay, it should be at its toe. To add to the stability of the repair, a strong earthen berm should be placed just beyond the original toe of the dam, and its own toe should be

secured by a second drystone wall. All these drains and walls should, if possible, be founded on rock, but, where this is not to be found at a reasonable depth, they should be carried well into the natural subsoil and beyond the limit of disturbance caused by the slip. Earthwork usually fails at one point, and, as soon as it commences to move, drags the adjacent parts with it. The drystone walls, instead of transmitting the pressure directly behind them, distribute it over a certain increased area, and thus tend to prevent the initial motion. They thus act like the timbering of a trench, which, although incapable of resisting the full lateral pressure of the earthwork, provides sufficient support to prevent the first tendency to slipping.

All excavation should be carefully taken out in sections, with sufficient widths of undisturbed material between them to act as buttresses, and should be filled as quickly as possible. This, of course, results in the filling not being so well compacted as if the whole length were dealt with at once, but in time this will be remedied. It also leads to the formation of numerous junctions, but, as the sections will be constructed within a short time of each other, the earthwork on each side of them will eventually unite in a practically solid way.

It will be thus seen that in dealing with a slip every means should be taken thoroughly to drain the earthwork and to cut out slipping planes and replace them by fresh material having the maximum amount of resistance to slipping.

XIII. Pitching.

152. The Necessity for Pitching.—In a large reservoir the action of the wind on the water surface will cause the formation of waves, which, if allowed to break on the unprotected surface of the dam, would very soon wash

away the earthwork. To protect that surface it has to be covered with pitching of a perfectly durable character. This is laid on a 6-inch layer of hard, sound murum, or quarry spauls, so as to prevent the entry of vermin and the growth of plants, and so that any water, which finds its way through the interstices of the pitching, may be prevented from washing out the clayey material of the dam, thus causing the pitching to subside and no longer to present an unbroken surface. In order to prevent subsidence of the dam from having the same destroying effect on the continuity of the pitching, it is desirable, whenever practicable, not to lay the stones until the dam has practically attained its final consolidation, for which reason the pitching should be constructed as late as possible.

In Appendix 21 are given tables for the estimation of the pitching of a dam embankment.

153. The Laying of Pitching.—The most economical form of efficient pitching consists in laying the stones with their broadest ends on the dam, and in hammer-dressing them roughly so that they meet for a depth of 3 or 4 inches all round their bases so as to form a complete covering. They should break joint in every direction, and long, continuous joints should be avoided, as they may tend to cause the stones to slip in sections. Owing to the irregularity of the stones the work should be carefully carried out and inspected. In some of the earlier Bombay dams the stones were laid with their broadest ends upwards, and their tops were made to fit each other so as to form an unbroken surface. This form presents a more regular appearance than does the more recent one, but, as in it each stone rests on a small base, it is liable to be displaced by the impact of waves.

Where roughly-squared stones are cheaply procurable, they can be laid in regular courses breaking joint with

each other, and this forms the best kind of drystone pitching.

After a section of the pitching has been laid, it should be tested by a heavy hammer to see that each stone is solidly bedded and firmly bonded with the rest. Each interstice which is left should be then filled by the largest sized chip practicable, each chip being driven well home by a hammer, and, thereafter, the pitching should be again tested.

The following are the defects to be avoided in pitching: larger base of stone laid uppermost; long, unbroken joints; stones set with the longest dimensions of their bases not parallel to the centre-line of the dam; sharp abutting edges involving large interstices; stones projecting irregularly; loosely-placed stones; partial bedding of base of stones; bad fitting of bases of stones; loose packing and incomplete filling of interstices.

154. The Extent and Thickness of Pitching.—The pitching should extend from 2 feet below outlet sill level to 2 feet above the level of the anticipated maximum wave-wash at high-flood level. Its thickness at different levels should vary with the height of the waves it has to withstand; this height depends upon the "fetch" of the wind, which is the maximum length of water surface over which the wind may pass. If the thickness in feet is taken as equal to one-third of the height of the waves, it would be :—

$$\tfrac{1}{3}\{1{\cdot}5\sqrt{F} + (2{\cdot}5 - \sqrt[4]{F})\},$$

where F = the "fetch" in miles.

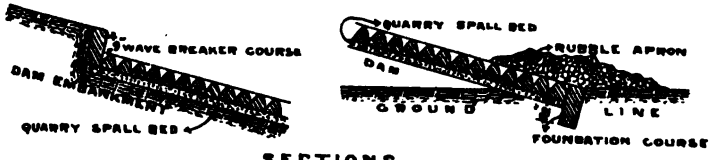
Practically, it would vary in the case of moderate sized reservoirs from 6 inches at the bottom to 18 inches at the top, and, in the case of the largest sized reservoirs, from 9 inches to 2 feet. In all cases the thickness must be formed by single stones of the full depth of the pitching, for, if it were made up of two layers, the upper layer, when

under the impact of the waves, would tend to be washed off from the lower one owing to the friction between the two being diminished by the lubrication of the water.

If the reservoir is to the leeward side of the dam in respect to the prevailing wind, the thickness of the pitching may be somewhat reduced, provided there is no chance of strong winds coming from the reservoir side.

155. Foundation and Top Courses of Pitching.—The foundation course of the pitching should consist of large stones sunk at least 12 inches into the dam or into the natural ground, and, where the toe of the dam is above outlet level and the soil consists of soft material, it should

FIG_18. FIG_19.



SECTIONS

be protected from being undermined by wave-wash by an apron of rubble *débris*, &c., from the various excavations connected with the main work (Fig. 19).

The top course should be formed of hammer-dressed headers with comparatively thin joints so as to prevent the upper part of the dam from guttering through them under the action of rain. This course should be laid in a continuous line parallel to the top of the dam so as to give a finish to the pitching, and should project at least 9 inches above it so as to act as a wave-breaker course (Fig. 18).

156. Concrete Packing of Pitching.—The form of dry-stone pitching described above is the cheapest one which is reliable, but it has not a very sightly appearance. It is liable to have interstices, in which the percussive action of the waves

may exert great force which will tend to blow out the stones, and through them the water will readily gain access to the surface of the dam and saturate it. These interstices may permit of the growth of plants near and above full-supply level, which may disturb the pitching, and they may allow of the entry into the dam of rats and crabs, which may be thus able to burrow and do a great deal of harm not noticeable from the surface. The projecting tops of the stones permit the waves to exercise a considerable amount of leverage in disturbing the pitching. The proper inspection of such pitching is at all times difficult.

These defects would be remedied were the pitching packed to a fairly uniform surface with fine concrete, or very coarse mortar, worked into all the interstices so as to unite the whole into a solid covering. To allow for settlement, this packing should be completed in detached sections, say 10 feet long and 5 feet wide, breaking joint with each other. The packing should be carried out a year or two after the reservoir has first filled, (by which time the earthwork of the dam will have practically attained its final settlement), and while the water level is falling, so as to permit, at small cost, of its being kept wet in order to set properly. This form of pitching would, of course, be dearer than ordinary dry pitching, but it would be more durable and sightly, and would be more easily inspected and maintained in good order.

157. Concrete and Brick Pitching.—On some works[1] in England pitching formed of concrete blocks has been tried, but in India this would be probably a much dearer form than stone, as the latter will be usually available at a cheap rate at dam sites. Moreover, concrete blocks will not be so durable as stone, and will be peculiarly liable to

[1] "Minutes of Proceedings Inst. C.E.," Vol. cxxxii., p. 226.

wear at the joints. It would seem better if concrete has to be adopted, to form it *in situ* in large slabs, say 10 feet long and 5 feet wide, and separated from each other by free joints in order that it may accommodate itself to the settlement of the dam without at the same time exposing numerous joints to the action of the waves.

Brick pitching has been used on some river embankments in Sind and might be tried on small reservoirs where stone is not available, being laid stretcher-wise for small and header-wise for moderate sized reservoirs. Bricks are scarcely hard enough to resist the impact of large waves, and cannot be made of sufficient depth to withstand it.

158. Grass or Reed Revetments.—In some Madras[1] tanks a grass, or reed, revetment is used. The reed employed is one that grows in marshy land, and has numerous joints, at each of which is an eye, from which a shoot will spring if it is buried in the ground. A layer of reed is first placed horizontally on the embankment with the root ends inwards; along the centre of this a fascine made of the reed is placed parallel to the axis of the embankment, and the rest of the layer is folded over it and is then covered with soil, and subsequently other steps are similarly formed. The whole is then watered until the eyes at the joints of the reeds begin to shoot. Eventually a forest of reeds, 10 or 12 feet high, springs up and forms an impenetrable barrier to the action of waves.

This is, of course, the cheapest form of protection possible, but it is open to the great objection that the surface of the embankment is entirely hid from inspection, and thus rats and crabs might burrow in it unperceived and unsuspected. It is an essential for a covering of an embankment of any height that it should be fully open to view, so that any

[1] Buckley's "Irrigation Works in India and Egypt," p. 83.

damage to it, or to the underlying earthwork, may be at once detected and repaired.

In Sind, river embankments are sometimes protected by allowing tamarisk to grow a little distance in front of them, and, in the case of high embankments, up the base of the slope, care being taken to trim off all bottom branches so that the earthen surface may be fully exposed to view.

Neither of these forms of protection by natural growth is, however, suitable to high reservoir embankments.

CHAPTER III.

THE WASTE-WEIR.

I. The Weir Proper.

159. Different Forms of Waste-Weir.—Waste-weirs are usually either of two forms :—

(*a*) Drowned weirs or channels ;

(*b*) Clear overfall weirs.

These two classes of weirs are simple and perfectly automatic in their action, and their selection in any one case depends upon the nature of the ground and upon its levels. For both classes the bed of the upstream approach channel is level, and practically at the same level (*vide* paragraph 164 below) as the actual crest of the weir, while the downstream tail channel is made with a slope falling from the weir crest, so as to provide for the passage of the flood with a small tail depth; the inclination of this slope is usually made 1 in 100.

The following table shows the discharging power per foot run of weir of the two classes :—

Discharge of Waste-Weirs per Lineal Foot in Cubic Feet per Second.

Total depth of Flood in Feet.	1	2	3	4	5	6	7	8	9	10
Drowned Channel with fall of 1 in 100 (approximate)	2·38	7·29	13·70	22·22	32·55	43·92	57·04	71·87	87·92	106·05
Clear Overfall Weir... ...	3·29	9·39	17·67	27·16	38·25	50·52	63·87	78·22	93·55	109·77

The drowned channel discharges are taken from

Appendix 10, and the clear overfall discharges, from Appendix 11.

160. Drowned Weirs and Channels.—A drowned weir is one of small height, having its crest below the surface of the tail channel discharge when the reservoir is at high-flood level.

A channel waste-weir is one where the crest of the weir is at its bed level. Where the natural surface is of rock, or hard material, and is at, or above, full-supply level, the channel can be left as a simple excavation. Where, however, there is any likelihood of erosion, the actual crest of the weir should be constructed as a wall founded on a reliable stratum so as to preserve the full-supply level of the reservoir, and so as to distribute the flood discharge evenly over the whole width of the tail channel. Without such a crest wall, one or more deep scour channels would be rapidly formed by the concentration of the tail discharge down them. The channel form with a gentle longitudinal slope is best suited to soft soils, so that the severe action of an overfall on them may be avoided.

161. Clear Overfall Weirs.—A clear overfall weir is one having its crest above the surface of the tail channel discharge when the reservoir is at high-flood level. As the action of the overfall on the foundations is very severe, the weir wall must be founded quite securely. The best foundation is one of solid, unfissured, compact rock, into which the waste-weir wall should be founded for at least 2 feet in depth. Where this material does not exist, the wall must have its foundations carried considerably deeper and at least to 5 feet in depth; and, in the case of soft, erodible soils, they must be protected by a water-cushion, which will also lessen the horizontal velocity of the water as much as possible.

Where the natural levels and formation of the ground

permit, it is advisable to convert channel weirs into clear overfall ones, so as to gain increased discharging power. It is, of course, not necessary for this purpose to lower the tail channel more than the depth of the high-flood discharge down it; in fact, where the ground is of a soft nature, a less amount of excavation will suffice, as the floods will themselves scour out the channel. The allowance for this scouring action can be safely made by deepening, to the full extent and full bed-slope, only the head of the tail channel for a length sufficient to prevent the surface of its discharge from topping the weir, the bed of the rest of the channel being continued therefrom with a less inclination.

In the case of a few old native tanks where the foundations are bad and the ground level is below the full-supply storage, the waste-weir has been formed as a rapid; but this has not been done in the case of modern works of any size, as rapids may entail much expenditure on maintenance. A natural rapid of hard material does not, however, involve as much expenditure on maintenance as does an artificial one, as excessive erosion of it can be prevented by cross curtain walls.

162. Positions of Weirs—Flank and Saddle Weirs.—In respect to their positions waste-weirs may be classed as:—

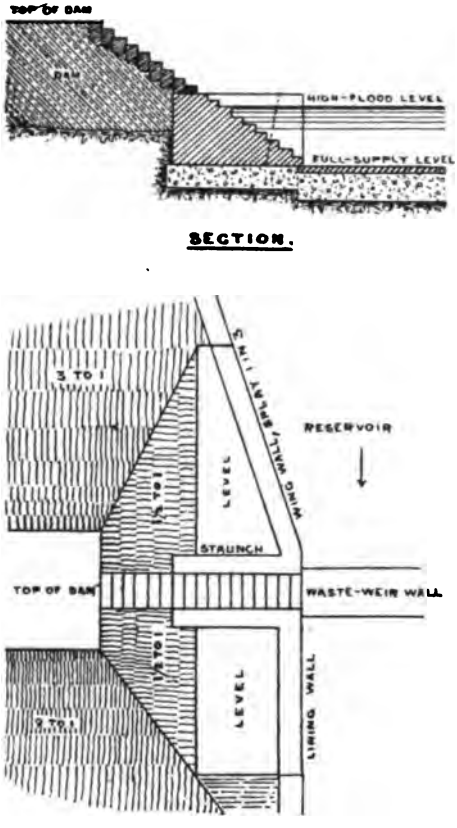
(*a*) Flank weirs, at the immediate flank and in continuation of the dam embankment;

(*b*) Saddle weirs, separated from the dam by high ground.

Flank weirs are naturally the less safe form of the two, as the discharge from them is liable to out-flank and injure the dam. To prevent this a wing wall is required on the upstream side and a lining wall on the downstream side. The lining wall is necessary to divert the floods from the dam; it should be continued until the tail channel has a clear course away from the embankment, and should be

raised at least 2 feet above the surface of the tail channel high-flood discharge. The junction of the waste-weir wall and the dam is best effected, as shown in Fig. 20, by a stepped passage, which permits of easy access to the works and at the same time acts as a staunching fork, preventing leakage from occurring at the junction.

FIG. 20.



Saddle weirs do not require such protective works, as they discharge clear away from the dam, but, to prevent them from being out-flanked, they should have masonry flanks on each side, raised at least 2 feet above high-flood level, and flood embankments in continuation, raised at least 5 feet above that level. As explained in Chapter II., paragraph 75, these flank embankments can be utilised as "breaching sections" to prevent the over-topping of the main dam embankment during abnormal floods. Similar flank works are required for flank weirs at the side remote from the dam, when the ground there is below high-flood level.

Saddle weirs are decidedly the better form, and should be adopted wherever practicable in preference to flank weirs, as the waste-weir tail channel should discharge as far away as possible from the dam.

163. Sections of Weirs.—There is no advantage, so far as discharging capacity is concerned, in raising the crest

FIG_21. FIG_22. FIG_23.

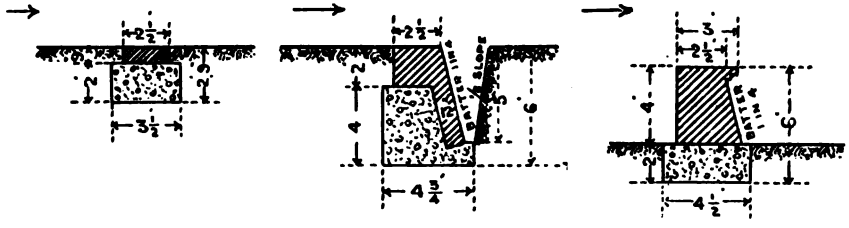
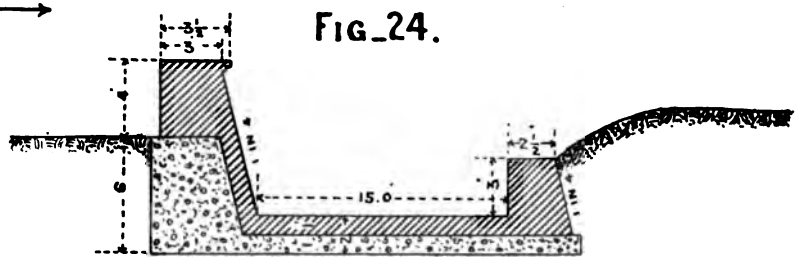


FIG_24.



SCALE 10 FT. = 1 INCH

of the waste-weir wall above the high-flood level of the tail channel, and such additional raising has the great disadvantage of increasing the action of floods on the foundations of the weir. Waste-weir walls will be, therefore, generally low, and will be high only when the ground where they are constructed is much below the designed full-supply level of the reservoir.

Fig. 21 shows a form which may be adopted for the crest

wall of a channel weir with good foundations; Fig. 22, one for a channel weir with bad foundations; Fig. 23, one for a clear overfall weir with good foundations; and Fig. 24, one for a clear overfall weir with bad foundations.

Where there are not to be any weir-crest shutters, the top-width for low weirs should not be less than 2 feet 6 inches; where such shutters are used, the top-width will have to be increased to permit of their being worked. The upstream face of the wall should be vertical, and its downstream one should batter 1 in 4. Preferably the downstream face of a weir with deep foundations should have the batter continued as a masonry facing to protect the concrete base. The crest should always be level in cross section, as it is not necessary to ramp it down to the reservoir side to help the passage of silt over it, as is desirable in the case of a river weir. The downstream edge of the crest should be corbelled out so that the floods passing over it may be made to discharge clear of the downstream batter of the weir. The mean width of weirs of a less total height than 10 feet may be taken in ordinary cases as equal to half their height: for weirs of greater height, and for those discharging a great depth of flood, their sections should be determined by means of stability diagrams.

164. The Approach Channel.—This is usually excavated level with the crest of the weir, but, where it is very long and the water in passing over it would thus lose head by the friction of the bed, it is preferable to excavate it from 6 inches to 12 inches lower than crest level.

The approach channel must have a perfectly clear and unobstructed course of the full width to the waste-weir, as any contraction of it would cause the water to head-up upstream, and would thus diminish the discharging capacity of the waste-weir crest. In determining that discharging capacity the effective length of the waste-weir crest to be

taken into account is that which is measured normally to the line of flow. It is of no use curving the weir crest in plan in order to increase its length beyond this amount, as that will not lower the reservoir below the level necessary at the narrowest part of the approach channel to pass the required discharge. Such an increase of length will diminish slightly the actual flood depth over the crest itself, but will thus induce a greater velocity of approach to it, and the head thus consumed will be equal to the diminution of the flood depth over the weir crest. Were this artifice effective, it could be further developed greatly by making the weir crest deeply serrated in plan, and this at once shows that it is not really of any use.

The amount of excavation of the approach channel of a flank weir may be, however, considerably decreased, where necessary, by curving it, as shown in Fig. 28, paragraph 177, as this will save the removal of the soil between the outer curve and the line at right angles to the end of the weir (where it will usually be of the maximum depth) without diminishing the normal width of the channel. Care must be taken, however, not to make the curve too abrupt, or else the high-flood water will have an irregular flow over the weir crest, which will thus be less effective in discharging it. For this reason the approach channel curve should be set out from a centre on the centre line of the dam some distance away from the waste-weir.

165. Safety Flood Cuts.—Where the ground at the proper level is sufficiently hard, auxiliary flood cuts can be excavated so as to help the discharge of the main weir in times of abnormal flood. The level to which these should be excavated is one which will let them come into play only during high floods, so that they will not be exposed to the long-continued action of moderate floods; they will thus not require protective works to prevent their erosion.

Generally speaking, that level may be 2 feet below the calculated high-flood level. Such flood cuts will not have a great discharging power per foot run, and, therefore, to make them of substantial assistance to the main weir they must be of considerable length. They should, of course, be situated where their discharge will not do any injury.

That discharge cannot be utilised for irrigation, partly because it will take place at such rare and uncertain intervals, and partly because, when it does occur, it will be the result of excessive rainfall, rendering irrigation unnecessary at the time. In the case of the main waste-weir, however, an irrigation channel might be led off by a pipe, somewhat below full-supply level and extending beyond the reach of the waste-weir discharge, when it can thus utilise for the irrigation of high lands water which would otherwise run to waste.

166. The Depth of the Maximum Flood on the Waste-Weir.—The depth of the maximum flood permissible on a waste-weir depends upon the nature of its foundations; the sounder they are, the greater may be the depth of flow. A deep discharge, of course, lessens the length of the waste-weir required, and may thus somewhat diminish its cost, as flank works will be generally cheap compared to the overfall portion. On the other hand, a long weir is safer than a short one, as any abnormal increase of the depth of the flood over it will produce a much greater discharging power than the same increase of depth will do over a short weir.

As explained in Chapter II., paragraph 78, the less the flood depth over the waste-weir, the lower will be the height required for the dam, and, consequently, the cheaper will be the embankment. Comparative estimates of the combined costs of waste-weirs of different lengths and dams of different corresponding heights should be, therefore, made,

and, other things being equal, the cheapest combination of the two should be selected.

The depths of the maximum floods generally allowed vary from 4 feet to 6 feet, and the weirs are, therefore, of great length. This margin of safety between full-supply level and high-flood level involves what may be considered either as a very expensive, but necessary, addition to the cost of the dam, or as a surrendering of a large amount of storage. In a few cases the temporary raising of the reservoir level by a small amount at the end of the monsoon has been attempted, either by means of low earthen banks, or by removable teak shutters fitted into removable rolled joist uprights. The length of the weirs makes this expensive, and, what is worse, slow in management, but the usual design of the works renders it difficult to arrange for automatic, or quick-acting, devices for discharging floods, and without them such temporary raising of the reservoir surface is attended by an undesirable amount of risk.

167. The Factors Producing the Maximum Rate of Run-off.—In Chapter I., paragraph 7, the principal factors influencing the amount of the yield from a catchment have been described. These factors have a similar influence in determining the amount of the maximum rate of run-off which may be expected. Each catchment has a maximum rate of run-off depending upon its local conditions, but, it may be stated as a general rule, that the smaller the catchment the greater will be the intensity of its run-off, other conditions being the same. Very small catchments may be subject to intense rainfall falling at the same time over their entire areas. As the catchments increase in size, the less likely are they to have such intense rainfall occurring simultaneously over their whole extent, and thus the average rate of run-off from them will diminish.

Very heavy falls of rain, however, are known to occur

over very large areas at the same time; these generally last for long periods, so that the flood discharges of the upper tributaries of the reservoir have sufficient time to arrive at the site of the dam simultaneously with those of the lower streams. These widespread and long-continued falls of rain are, however, usually of less hourly intensity than that of sudden storms, but their effect is to make the rate of the maximum run-off from large catchments vary less in proportion to their size than that from small catchments.

168. Formulæ for the Rate of Maximum Run-off.—There are several formulæ for the rate of maximum run-off; the best known and most frequently used are described below.

(*a*) *Dickens' Formula.*—This formula is :—

$$D = C M^{\frac{3}{4}},$$

where D is the flood discharge in cubic feet per second;
M, the catchment area in square miles; and
C, a coefficient varying from 150 to 1,000 or more, and generally taken as 825.

This formula allows for the relatively greater discharge from small than from large catchments, and agrees fairly well in this respect with the table of waste-weir runs-off in Appendix 8, as will be seen by the following comparison :—

1	2	3	4	5	6
M = No. of square miles in catchment.	$M^{\frac{3}{4}}$	Relative Run-off per square mile.	Comparative Run-off per square mile.	Allowance for Run-off in Appendix 8.	Remarks.
			In inches per hour.	In inches per hour.	
1	1·00	1·00	3·00	3·00	Col. 3 = Col. 2 / Col. 1
5	3·35	0·67	2·01	2·36	
10	5·62	0·56	1·68	1·95	Col. 4 = Col. 3 × 3, in order to make its initial run-off allowance the same as that of Col. 5.
20	9·45	0·47	1·41	1·51	
50	18·79	0·38	1·14	1·10	
100	31·62	0·32	0·96	0·91	

(*b*) *Ryves' Formula.*—This formula, which is much used in Madras, is :—

$$D = C M^{\frac{3}{4}},$$

where D is the flood discharge in cubic feet per second ;

M, the catchment area in square miles ; and

C, a coefficient which is taken thus :—

within 15 miles of the coast.... = 450
from 15 to 100 miles from the coast.... = 563
for limited areas near the hills = 675

This formula also allows for the relatively greater discharge from small than from large catchments, but does not agree so well in this respect as does Dickens' with the table of waste-weir runs-off in Appendix 8, as will be seen by the following comparison :—

1	2	3	4	5	6
M = No. of square miles in catchment.	$M^{\frac{3}{4}}$	Relative Run-off per square mile.	Comparative Run-off per square mile.	Allowance for Run-off in Appendix 8.	Remarks.
			In inches per hour.	In inches per hour.	
1	1·00	1·00	3·00	3·00	Col. 3 = Col. 2 / Col. 1
5	2·92	0·58	1·74	2·36	
10	4·64	0·46	1·38	1·95	Col. 4 = Col. 3 × 3, in order to make its initial run-off allowance the same as that of Col. 5.
20	7·37	0·37	1·11	1·51	
50	13·55	0·27	0·81	1·10	
100	21·53	0·22	0·66	0·91	

(*c*) *Craig's Formula.*[1]—This formula takes into account the varying width of the catchments, their slopes, and the amount of the rainfall (but not its hourly intensity, which is a very important factor), and is therefore theoretically much more exact than the other two. It is :—

$$D = 440\ B\ (C\ v\ i)\ \text{hyp. log.}\ \frac{8\ L^2}{B},$$

[1] "Minutes of Proceedings Inst. C. E.," Vol. lxxx., p. 201.

where D is the flood discharge in cubic feet per second ;
B, the mean width of the area in miles ;
C, the coefficient of discharge, varying with the nature of the basin ;
v, the velocity of the drainage in feet per second ;
i, the rainfall in inches ;
L, the length of the catchment in miles measured from the point of discharge to the centre of the watershed base.

For irregular catchments, the perimeter has to be made regular by straight lines, and the whole area has to be divided into triangles with their apices at the point of discharge. The total discharge is the sum of the discharges of the component triangles.

169. The Limited Utility of Run-off Formulæ—Flood Observations—Cyclones.—It will be seen that each of these formulæ requires the application of a coefficient C, and this has to be determined in each case by experiment, so that the utility of the formulæ is much lessened. If there is a neighbouring catchment to that for which a project is contemplated, the flood discharges of which have been observed for a long series of years and the factors of run-off are precisely the same in the two cases, the results of the former may be applied to the latter by means of the selected formula. If, however, those factors differ, so also must the coefficients, and the results of the first catchment cannot be applied directly to determine the probable run-off until the proper coefficient for the second catchment has been ascertained.

The waste-weir has to discharge safely the maximum flood which may occur, although such a flood may not happen except at intervals of many years. It is therefore necessary to determine, as accurately as possible, the discharge of the maximum flood known. If there are good

records extending for at least twenty-five years (which will rarely be the case in India), they may be taken into account, but, where they do not exist, recourse must be had to local information. The height to which an abnormal flood rose will be generally impressed on the minds of the villagers, and usually they will be able to point out high-flood marks along the course of the stream. From these sufficiently accurate flood sections can be obtained, and the high-flood discharge can be thus calculated.

In order to provide for contingencies, it will be advisable in any case to allow 25 per cent. over the recorded, or calculated, flood when estimating the discharge for which provision should be made. It is far safer to rely upon these results of observation than upon a mathematical formula, for the intensity and quantity of the rain are not fixed quantities, and upon them the variation in the amount of the flood discharge of any particular catchment entirely depends. In the case of a reservoir, however, it is not necessary, as in that of a drainage work, to provide for the absolutely maximum rate of flood which may occur, for the storage capacity of the tank will act as a moderator of the intensity of the discharge, as will be explained in paragraphs 182—184 below.

Where drainage areas are liable to be visited by cyclones, it is not practicable to determine exactly what will be the discharge resulting from them, as their intensity is so abnormal. In such cases it is best to provide liberally for the ordinary maximum floods, and, in addition, to allow for these abnormal ones by designing breaching sections (Chapter II., paragraph 75) and safety flood cuts (paragraph 165 above).

In Chapter I., paragraph 28, it has been stated that in twenty Bombay tanks the average cost of the waste-weir

is only 9·44 per cent. of that of the whole reservoir. As a proper provision for this essential to safety is comparatively so cheap, and as the results of the bursting of a dam may be so disastrous, it is always more prudent to err on the side of safety and to make ample allowance for floods than to run any risk of failure by having insufficient waste-weir discharging capacity.

170. The Objection to Tanks in Series. — In this connection it may be noted that it is most undesirable to construct a chain of tanks one below the other on the same stream, for the failure of an upper one may cause all the lower ones to breach successively by what, in effect, will be an artificial cyclone. It may be said that modern reservoirs are constructed so as not to breach, which is, of course, true ; but accidents to them have occurred, and it certainly does not appear to be sound engineering to run any needless risks in this way. The proper treatment of such a stream is to form on it one reservoir capable of impounding a storage equivalent to that of all the proposed minor tanks. Where, however, from physical conditions a chain of tanks is absolutely unavoidable, care should be taken that the capacities of the works increase in the order in which they stand with relation to each other, *i.e.*, the downstream ones should always have larger storages than the ones upstream of them, so that the former may be better able to absorb the sudden and large inflow due to the failure of the latter. Full provision for dealing with abnormal floods should be made, in such cases, by means of breaching sections and safety flood cuts.

171. Empirical Allowances for Waste-Weir Runs-off. —Although it is better to depend upon observations for the determination of the maximum discharge for which waste-weir provision has to be made, still in India sufficient experience has been obtained of the behaviour of reservoirs

to enable allowances to be prescribed which err sufficiently on the side of safety, and can be therefore adopted without incurring risk.

A table of such allowances for average catchments is given in Appendix 8. The hourly rates of run-off from gradual increments of catchment area (*vide* col. 2) have been adjusted, so that, when plotted in diagram form (*vide* Plate 2), they are on a regular curve; the discharges due to these runs-off for each increment in catchment area have been entered in col. 3; the discharges from the total catchment areas have been given in col. 4, and from them the average hourly rates of run-off from the total catchments have been deduced and noted in col. 5 and plotted on Plate 2. The object of thus determining these average runs-off is to prevent a total discharge from a larger catchment being calculated from the table as less than that from a smaller one, and to obviate the neglect of the effect of the intensity of the rainfall on the smaller constituent areas near the dam. Thus, if col. 2 alone were considered, the amount of run-off from 10 square miles would be equal to $(10 \times 1{\cdot}40 =)$ 14·00 inches from 1 square mile, and from 11 square miles only to $(11 \times 1{\cdot}16 =)$ 12·76 inches from 1 square mile. Taking col. 5 properly into account, the results are, respectively:—

$$(10 \times 1{\cdot}95 =) 19{\cdot}50 \text{ and } (19{\cdot}50 + 1{\cdot}16 =) 20{\cdot}66$$

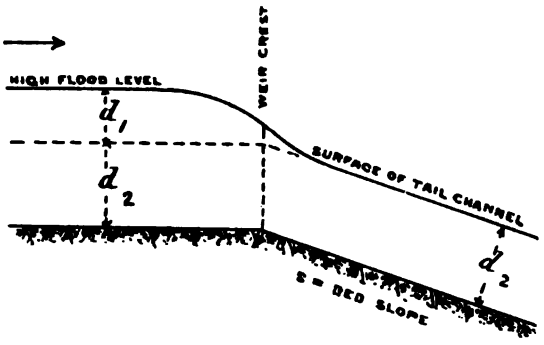
inches from 1 square mile.

Special conditions of the catchment area, of course, have to be taken into account; a purely ghát catchment would be given a greater, and a purely plain one a smaller allowance. Allowance for the general intensity of heavy rainfall in the locality and also for the amount of actual floods would have to be made.

172. Formulæ for the Discharge of Waste-Weir

Channels.—For calculating the discharge of these the following formulæ are used (*vide* Fig. 25) :—

FIG. 25.



(i.) $D = a\sqrt{r} \cdot c_2\sqrt{s}$. . . (Chézy) ;

(ii.) $D = c_1 b \sqrt{2g} d_1 \left(d_2 + \frac{2}{3}d_1\right)$. (Eytelwein) ;

$$\text{or } \frac{D}{c_1 b\sqrt{2g}} = \sqrt{d_1}\left(d_2 + \frac{2}{3}d_1\right),$$

where D is the discharge in cubic feet per second ;

a, the cross-sectional area of the flood in square feet ;

r, its "hydraulic mean radius" = $\frac{\text{area (in square feet)}}{\text{wetted perimeter (in feet)}}$;

c_1, the afflux coefficient (*vide* Appendix 10) ;

c_2, Bazin's coefficient for the tail channel (*vide* Appendix 9) ;

s, the slope of the tail channel = $\frac{\text{fall}}{\text{length}}$;

b, the breadth, or length, of the weir in feet ;

d_1, the height of the flood afflux in feet ;

d_2, the depth of the tail channel discharge in feet ; and

g, the force of gravity.

D and s being given, then a ($= b \times d_2$) is found by trial

in the first formula. From the second formula d_1 is ascertained, generally by trial to avoid a cubic equation. The height to which the flood will rise in the reservoir is $d_1 + d_2$, and this must be provided for, care being taken not to neglect d_1, as is sometimes erroneously done.

173. Calculation of the Discharge of a Waste-Weir Channel.—An example of the calculation of the discharge of a waste-weir channel is given below. The channel is assumed to be 200 feet wide, with vertical sides and a longitudinal slope from the crest line of 1 in 100, and the total flood depth to be 8 feet. It is in the first place necessary to assume both the tail depth and the afflux height, and to make trial calculations until the correct results are obtained.

A. *Tail Depth.*

Assume $d_2 = 4{\cdot}48$ feet $\quad a = bd_2 = 200 \times 4{\cdot}48 = 896$

$D = a\sqrt{r} \cdot c_2\sqrt{s}$
$= 896 \times 2{\cdot}07 \times 77{\cdot}5 \times 0{\cdot}1$
$= 14{,}374$ cusecs

$r = \frac{a}{wp} = \frac{896}{209} = 4{\cdot}29$
$\sqrt{r}$ $= 2{\cdot}07$
c_2 $= 77{\cdot}5$
$s = 0{\cdot}01$; $\sqrt{s} = 0{\cdot}1$

B. *Afflux Height.*

Assume $d_1 = 3{\cdot}52$ feet.

$$\frac{D}{c_1 b\sqrt{2g}} = \frac{14{,}374}{0{\cdot}7 \times 200 \times 8{\cdot}02} = \frac{14{,}374}{1{,}123} = 12{\cdot}80$$

$$\sqrt{d_1}\left(d_2 + \frac{2}{3}d_1\right) = \sqrt{3{\cdot}52}\left(4{\cdot}48 + \frac{2}{3}(3{\cdot}52)\right)$$
$$= 1{\cdot}88 \times 6{\cdot}83 = 12{\cdot}84$$

Therefore the two sides of equation (ii.) are approximately the same with the assumed values, and the assumptions are sufficiently correct. The total flood depth is:—

$$d_1 + d_2 = 3{\cdot}52 + 4{\cdot}48 = 8{\cdot}00 \text{ feet.}$$

In calculating $\sqrt{r}$ and $\sqrt{d_1}$ a table of square roots will be found most useful. Appendix 10 gives the values of c_1, and Appendix 9 those of c_2.

It may be noted, as an aid to adjusting d_1 and d_2, that for any given change of d_1 the value of the side of the equation $\sqrt{d_1}\,(d_2 + \frac{2}{3}\,d_1)$ will change less than will the side of the equation $\frac{D}{c_1\, b\, \sqrt{2g}}$ for the corresponding variation of d_2 required to make the sum of $d_1 + d_2$ the same. This is illustrated by the following examples:—

1	2	3	4	5	6	7	8	9
Factors.	Case 1.			Case 2.				
d_1	5·00	5·10	5·20	7·00	7·10	7·20	7·25	7·30
d_2	6·00	5·90	5·80	8·00	7·90	7·80	7·75	7·70
$d_1 + d_2$	11·00	11·00	11·00	15·00	15·00	15·00	15·00	15·00
$\sqrt{d_1}\left(d_2 + \frac{2}{3}d_1\right)$	20·90	21·02	21·14	33·58	33·68	33·77	33·84	33·94
$\frac{D}{c_2 b \sqrt{2g}}$	21·47	20·87	20·28	34·63	33·88	33·12	32·78	32·55

Columns 3 and 6 show the nearest correct approximations possible, taking only the first place of decimals into account for d_1 and d_2, and greater accuracy is not necessary in such calculations.

174. The Discharge of a 200-foot Waste-Weir Channel.—A table of the discharges of a 200-foot waste-weir channel with a tail slope of 1 in 100, and with total flood depths varying, foot by foot, from 1 foot to 20 feet, is given in Appendix 10. As the discharges vary with the extent of the flood's wetted perimeter, the discharges of weir channels with the same depth are not in exact proportion to their different widths. However, as the wetted perimeters of such floods very nearly vary with their widths, for all practical purposes the discharges of the floods may be taken as proportionate to their widths, and especially is this the case for the small depths which

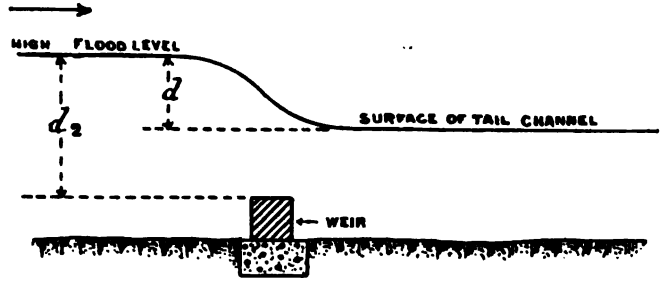
usually have to be considered. Examples bearing this out are given below :—

Width of channel ...	Feet	1,000	1,000	50	50
Total depth of flood...	,,	2	10	2	10
By direct calculation	Cusecs	7,322	106,555	362	5,180
Deduced from Appendix 10	,,	7,285	106,050	364	5,302

This table will be found very useful in determining the width of the weir channel and the total flood depth to be adopted in any particular case.

175. Formula for the Discharge of Drowned Waste-Weirs. — For calculating the discharge of these the formula[1] used is (*vide* Fig. 26) :—

FIG. 26.



$$\text{(iii.)}\quad D = c_1 b \sqrt{2gd}\left(d_2 - \frac{d}{3}\right),$$

where D is the discharge in cubic feet per second ;

c_1, a coefficient, which may be taken as in Appendix 10.

b, the breadth, or length, of the weir in feet ;

d, the height in feet of the surface of the afflux above that of the tail channel ;

d_2, the height in feet of the afflux above the weir crest ; and

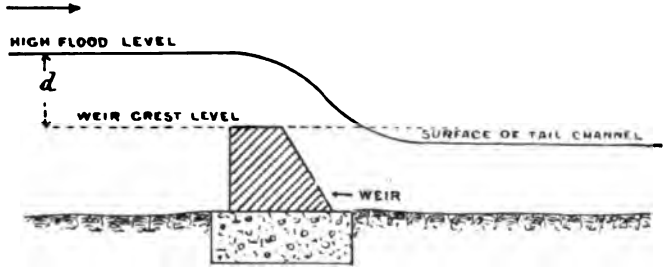
g, the force of gravity.

[1] Prof. Unwin's Article on Hydromechanics in the "Encyclopædia Britannica," Vol. 12, p. 473, 9th Edition.

The calculation of the discharge is a straightforward one, and an example of it is therefore not worked out.

176. Formula for the Discharge of Clear Overfall Waste-Weirs.—For calculating the discharge of these Francis' formula [1] (*vide* Fig. 27) is used, namely :—

FIG. 27



$$\text{(iv.)}\quad D = \frac{2}{3} cbd\sqrt{2gd},$$

where D is the discharge in cubic feet per second;

c, the coefficient (*vide* Appendix 11, note 3);

b, the breadth, or length, of the weir in feet;

d, the height in feet of the afflux above the weir crest; and

g, the force of gravity.

The calculation of the discharge in this case is also a straightforward one, and an example of it is therefore not worked out.

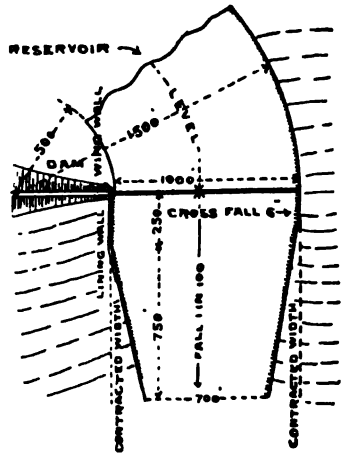
In all these formulæ the surface of the afflux is that of the still-water level of the reservoir, or, in other words, the afflux surface is its high-flood level.

[1] Prof. Unwin's Article on Hydromechanics in the "Encyclopædia Britannica," Vol. 12, p. 472, 9th Edition.

II. The Tail Channel.

177. The Section of the Channel.—Not only must the flood discharge have a clear approach to the waste-weir, but also it must have a perfectly unobstructed exit from it, or the obstructions may cause it to head up, and may thus make it mask and interfere with the discharging power of the weir. If, however, the contraction of the tail channel will not cause the flood to head-up as far as the weir crest itself, it will, of course, not interfere with its discharging capacity. The amount of excavation of a long tail channel can therefore be considerably reduced by gradually contracting its width, as shown in Fig. 28.

Fig. 28.



Another advantage of thus contracting the channel is that the floods, instead of being distributed over a very wide bed, will be confined to a comparatively narrow cut, and will thus tend to keep to a definite course and not to spread over a large area; when they might perhaps excavate for themselves irregular scour channels, which might cause injury to culturable lands.

The channel should start from the waste-weir with the calculated bed fall. If this involves a large amount of excavation, the bed fall may be reduced as soon as such reduction can be effected without causing heading-up at the weir crest. Where, however, the natural fall of the country exceeds that of the calculated bed fall, the channel may be

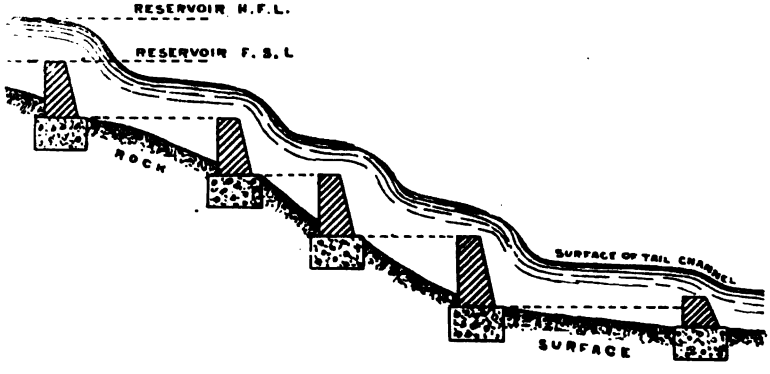
given as steep a slope as the former, provided that thereby excessive erosion is not caused up to the waste-weir itself. The tail channel should be led as soon as possible into a natural drainage line, so as to diminish the amount of excavation and so as to secure a definite course for the floods without the necessity for protective embankments. The flood water should be confined to the tail channel by means of lining walls, pitched slopes, or flood embankments, until it can no longer cause injury to the reservoir works or to cultivated lands, buildings, &c.

In order to direct the flood away from the dam, in the case of flank weirs it is desirable to give the tail channel a small cross-sectional fall—say of 6 inches—to the side remote from the dam. As a general rule in flank weirs the inclination of the natural ground will be towards the dam, and this should be counteracted by this artificial cross-sectional fall in the opposite direction given to the tail channel.

The above remarks apply to channel and to drowned weirs. In the case of clear overfall weirs little or no excavation of the tail channel will usually be necessary, as the flood waters will find their own way down the natural depression which will generally exist below the centre of the weir. Also, in the case of saddle weirs, when they are remote from the dam, the floods from them cannot injure the works, and it is usually cheaper to pay compensation for the damage they may cause to the fields through which they pass than to construct embankments or to excavate channels to protect them. It may, however, be useful, even in their case, to excavate a shallow channel so as to direct the first floods; the floods will be thus led at but little expenditure to scour out a deep cut down to hard material, and thus may effectually confine themselves to a definite course.

178. "Retrogression of Levels."—A tail channel does not usually widen itself, although severe floods passing down it before it has cut a definite channel for itself may cause the formation of scour channels branching from the main one. The effect of floods is generally to make the tail channel deepen itself, and the deepening as a rule is effected by the cutting back of the bed where the soil is soft; the exposed face, even when of hard material, may be thus eroded

FIG. 29.



SECTION OF WATER CUSHIONS IN SERIES

away with rapidity: this action has been termed the "retrogression of levels." As long as the erosion does not extend to the weir itself, it does no harm, but, if it threatens to reach it, it must be prevented by the construction of curtain walls built right across the tail channel and founded securely, and, if possible, in rock. Where hard material does not exist in the channel bed, these walls must be protected by pitched rapids, or, better still, by water-cushions.

This retrogression of levels is most certain to occur at a site where the waste-weir is situated on a ridge with a

steep fall downstream, as even the soundest rock is liable to be detached in large blocks by the rapid descent of floods over it. The best treatment of such a site is to form a series of water-cushions down to the general level of the country, as each of them will destroy most of the horizontal velocity of the water and will deliver it with a comparatively gentle flow below the lowest one (Fig. 29).

The great advantage of curtain and water-cushion walls is that they distribute the floods evenly over the bed of the tail channel downstream of them and thus prevent the formation of scour channels, which might rapidly deepen with each flood, since the discharge will be concentrated down them.

179. The Effect of the Bed Slope upon the Velocity of the Tall Channel Discharge.—The velocity of the flood down a tail channel is calculated by the formula (*vide* paragraph 172 above) :—

$$V = c_2 \sqrt{r s},$$

where V is the velocity in feet per second ;

c_2, Bazin's coefficient (Appendix 9) ;

r, the hydraulic mean radius = $\frac{\text{area (in square feet)}}{\text{wetted perimeter (in feet)}}$; and

s, the slope of the channel = $\frac{\text{fall}}{\text{length}}$.

The effect on $\sqrt{s}$ of increasing the slope is at once apparent from the figures given below :—

Bed Slope, 1 in	5,280	2,640	1,320	1,000	500	200	100
s =	0·000189	0·000379	0·000758	0·001	0·002	0·005	0·01
$\sqrt{s}$ =	0·014	0·019	0·028	0·032	0·045	0·070	0·100

In other words, $\sqrt{s}$ is seven times as great when the slope is 1 in 100 as when it is 1 foot in a mile. An

increase of s will, however, diminish r and c_2, but on the whole will increase V.

The slope to be selected depends chiefly upon the nature of the bed of the tail channel, as the velocity produced by it should not exceed that which the soil can stand without excessive erosion. Where the channel is in rocky ground, a slope of 1 in 100 can be given as a maximum ; where it is in hard ground, the slope should not exceed 1 in 500 ; and where it is in ordinary soil, it should not be steeper than 1 in 1,000.

The effect of reducing the inclination of the slope while maintaining the weir of the same length is, of course, by lessening the velocity, to increase the depth of the tail flood, and, thus in the case of channel and drowned weirs, to raise the afflux height and to add to the cost of the dam which has to be constructed correspondingly higher. Where the tail slope has to be made flat, it will be generally found cheaper to lengthen the waste-weir rather than to raise the dam, but the decision as to what has to be done in each case should be arrived at after making alternative estimates of the two works combined. It will usually be cheaper to reduce the velocity of the tail channel discharge than to depend upon protective works to control it. Too high a velocity may be dangerous, and too low a one may entail unnecessary expense.

Where a weir has its outfall into a stream parallel to itself and with a bed considerably below its crest level, the weir should, when the ground is soft, be placed as far away from the stream as is necessary in order to reduce the inclination of its tail channel ; or, if this is not practicable, a good water-cushion should be formed below the weir. If, however, the bed of the stream is rocky and can withstand the overfall, the weir may be advanced towards it.

180. The Outfall of the Tail Channel.—The most

desirable outfall for a waste-weir is one into the stream on which the reservoir is constructed, and as near to the dam as is practicable without injuring the works or interfering with their drainage. This latter is a most important consideration, as the floods, when cutting out a new channel for themselves, will bring down a large amount of detritus, which will tend to choke the bed of the main stream and to block channels excavated in it for drainage.

Where the outfall is to another stream, unless the channel of that stream is ample to pass off the added drainage, (which will seldom be the case in India, where heavy floods usually overflow the banks), damage may be caused to the neighbouring lands. The bed, too, of the original stream being deprived of scour, may silt up, thus interfering with the drainage of the dam; may become marshy and covered with rank vegetation; and may thus produce malaria. The subsoil water level of the riparian lands may be raised, and perhaps a saline efflorescence may be formed, thus rendering these fields unculturable.

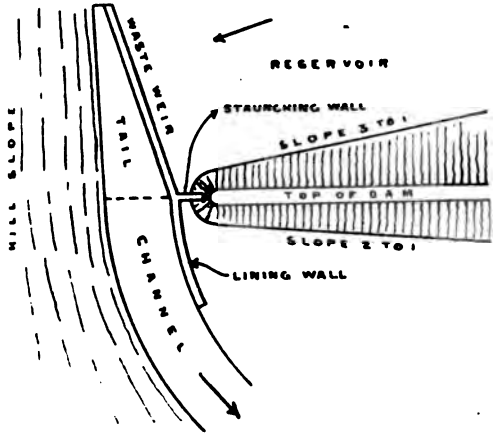
181. Design with the Tail Channel Parallel to the Waste-Weir.—In the previous paragraphs tail channels at right angles to the waste-weir have been discussed. Such channels are in general use, but another form is practicable in which the channel at starting is parallel to the waste-weir, and this is illustrated in Plate 4, Fig. 1, and Plate 6, Fig. 2. In the case of the Máládevi tank project the parallel channel was adopted to lessen excavation and to get a tail discharge in a definite channel suitable to the general design of the weir with its under-sluices and automatic gates.

This form is also adaptable to sites where the waste-weir accommodation is naturally restricted in extent by high ground. This is illustrated in Fig. 30, where a high spur is shown coming close to the dam. The waste-weir is therefore designed nearly parallel to the high-flood

contour of the spur, and at a distance from it sufficient for the space required by the tail channel. It will be generally best in such a case to build the weir as a clear overfall one, but, where necessary, it may be of the channel form, the tail channel being then excavated as a trench parallel to the crest of the weir.

In calculating the sections of such a parallel tail channel, its water surface slope must be first determined, as its discharge depends upon this, and not upon its bed slope; the steeper this surface slope, the smaller will be the sections of the tail channel. The area required to enable the channel at any point to carry the discharge due to the length of the weir upstream of it can be calculated in the usual way. The channel will be thus increased in width gradually from the upstream to the downstream end of the weir, and will be continued from it with its maximum width until this can be safely reduced, as explained in paragraph 177 above. In making the calculations, account must be taken of the losses of velocity due to the tail water having at once to change its direction through a right angle and to the effects of eddying motion. It is believed no such form of tail channel has yet been constructed for a large Indian flood, and it is therefore

FIG.-30.



advisable to make full provision for these losses when this design is adopted.

III. The Flood-absorptive Capacity of Reservoirs.

182. The Regulating Power of Lakes.—During high floods the surface of a reservoir rises because the waste-weir at the lower levels is not able to discharge the water at the same rate as that at which it is entering the tank. When the flood decreases, the discharging capacity of the waste-weir at the level attained by the reservoir being then greater than the rate of inflow, the reservoir surface is gradually lowered. The action of a reservoir is, therefore, to moderate the intensity of a flood by absorbing part of it as it arrives and to discharge this part after the flood has diminished.

The property of flood-absorption and discharge-regulation possessed by large lakes is well known, but it is believed that, at present, advantage of it has not been taken in the design of reservoirs constructed in India. The best known natural instances of it are the great lakes of North America and the Lake of Geneva. It is probable that the size of these lakes compared with that of their inflowing rivers is relatively larger than is the case with Indian reservoirs, and also that the maximum falls of rain filling them are of less intensity than those which occur in India, so that the rise of the storage during heavy storms is less there than it would be in the case of Indian tanks.

183. The Necessity for Large Absorptive Capacity.—Properly to utilise this property of regulation it is essential that the storage capacity of the reservoir between its "restricted" (*vide* paragraph 184) or full-supply level and its high-flood level must be large in comparison with the yield from the drainage area. Otherwise, a prolonged storm of great intensity may fill the tank to the last-named level,

and, if it continues thereafter, there will not be any flood regulation possible, and the waste-weir at high-flood level must, therefore, have a discharging capacity equal to the maximum rate of inflow. It is for this reason that in existing reservoirs advantage has not been taken of this property. In their case, it is seldom that the calculated high-flood depth over the weir exceeds 6 feet, that depth being fixed as a limit so as not to have a discharge of great volume per foot run over the weir and down the tail channel. There have been instances in Bombay of actual floods exceeding the calculated high-flood level when that was less than 6 feet, although, when the waste-weir discharge was calculated, the flood-absorption of the reservoir was not taken into account. It would therefore appear advisable not to make an allowance for the influence of the reservoir in regulating the flood discharge when the calculated high-flood depth over an ordinary weir is less than 6 feet.

It may, of course, be said that these instances of excessive flood prove that the rate of maximum run-off was underestimated, as, during the rise of the reservoirs up to calculated high-flood level, the rate of inflow must greatly have exceeded the calculated maximum discharging power of the waste-weirs. The flood-absorption during this period may, however, be considered as a provision for safety over and above the discharging capacity of the weir.

184. Method of safely utilising the Property of Flood-Absorption.—As it is not prudent to increase the depth of the maximum flood over a waste-weir, another plan must be adopted for utilising the flood-absorptive property of the reservoir if advantage of it is to be taken. This plan consists in tapping the reservoir at a low level, by "restricting" it to a low level before the maximum flood discharge arrives, so that that flood will have to raise the reservoir surface to a considerable extent; while that

surface is being raised the tank will absorb a large part of the run-off. During this period the rate of outflow from the reservoir will be less than the actual high-flood discharge of the river, and the natural intense flood of short duration will be thus converted into a prolonged one of less volume. The action on the waste-weir channel of the prolonged, gentle discharge will be less than that of the short, intense flood.

It must be remembered that the rise of the reservoir surface during a flood is very much more gradual than that of the inflowing stream, and that the maximum discharge of that stream is produced long after the rain has commenced to fall. There is thus ample time from the beginning of a heavy storm to lower the tank to a considerable extent if sufficient provision has been made in the design of the waste-weir to permit of the escape of a large discharge from the reservoir at a low level. A design which will thus safely utilise the flood-absorptive capacity of the reservoir is described in paragraphs 188 and 191 below.

185 The Allowance for Flood-Absorption in the Reservoir.—In order that the provision for flood-absorption in the reservoir may be sufficient to prevent its calculated high-flood level from being exceeded, full allowance must be made for the total yield from the maximum flood which may be expected from the catchment. Sir Thomas Higham has discussed this subject in a note, dated 24th March 1902, on tank projects and the method of dealing with waste-weirs in the Central Provinces. He lays down the following rules in the two cases considered:—

(*a*) *When the Waste-Weir alone is taken into account.*

"Unless the absorptive capacity of the tank above crest level is greater than half the total influx during the continuance of the flood, (which is not ordinarily the case), the

waste-weir must be designed to pass the whole run-off per second, and the flood-absorbing capacity of the tank above crest level must be neglected."

(*b*) *When the combined effects of the Waste-Weir and Outlet are taken into account.*

"The absorbing capacity of the tank must be neglected unless this will be greater than half the total inflow during the continuance of the flood less half the amount that would pass through the outlets during the same time, assuming the rate of outflow to be that obtaining at commencement of flood, or when the water surface in the tank is at crest level."

These rules have been based on mathematical considerations, but, as the intensity and duration of a maximum storm cannot be predicted, the following rules based on practical considerations are suggested :—

(*c*) *Rules for the provision of the proper extent of Reservoir Flood-absorptive Capacity.*

(i.) During the rise of the water surface in the reservoir, the flood disposed of hourly (by absorption in the tank and by the discharge from the waste-weir and the outlet) should be at least equal to half the calculated maximum rate of run-off from the catchment.

(ii.) During the total rise of the water surface in the reservoir to calculated high-flood level, the total flood disposed of should be at least equal to the yield of the calculated maximum flood from the catchment.

(iii.) The combined discharging capacity of the waste-weir and of the outlet at high-flood level should be at least equal to half the calculated maximum rate of run-off from the catchment.

It will be noticed that these rules (*c*) provide for the

effect of the capacity of the reservoir relative to the size of the catchment and also take into account the anticipated rate of run-off and the yield from it during heavy rainfall. They also provide a considerable margin of safety for abnormal floods by implying that the difference between the "restricted" level of the tank at the commencement of the flood and the calculated high-flood level shall be a large one.

186. Examples of Reservoir Flood-Absorption Calculations.—The flood calculations of the Máládevi tank project are selected as examples of calculations on the basis of rules (*c*). That tank has a catchment of 153 square miles, for which, according to Appendix 8, the average rate of run-off should be 0·82 inch per hour. As the drainage area is a good one for producing discharge, the rate of run-off might, however, be taken as 1·00 inch per hour.

Appendix 12 gives a calculation for the temporary waste-weir, first closure. During the rise of the tank, the run-off is disposed of at a rate varying from 0·43 to 0·68 inch per hour, and the total run-off dealt with in this period is 4·75 inches. The high-flood discharging capacity of the weir is at the rate of 0·546 inch per hour.

Appendix 13 gives a calculation for the temporary waste-weir, second closure. During the rise of the tank, the run-off is disposed of at a rate varying from 0·40 to 0·62 inch per hour, and the total run-off dealt with in this period is 3·98 inches. The high-flood discharging capacity of the weir is at the rate of 0·42 inch per hour, which is somewhat small, but the work as it would be constructed (*vide* the note to the Appendix) would give larger results.

Appendix 14 gives a calculation for the permanent waste-weir. During the rise of the tank, the run-off is disposed of at a rate varying from 0·44 to 0·75 inch per hour, and the total run-off dealt with in this period is 6·96 inches. The high-flood discharging capacity of the

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weir is at the rate of 0·418 inch per hour, and to this has to be added the discharge of the outlet, at the rate of 0·07 inch per hour, giving a total high-flood discharging capacity of 0·488 inch per hour.

IV. The "Stepped Waste-Weir."

187. Objections to the Usual Form of Level Waste-Weir.—The usual form of a solid weir wall with a permanent crest raised throughout to full-supply level is best suited for reservoirs the replenishment of which is not certain, for, in such cases, the storage level has to be kept as high as possible during the whole of the monsoon, so that full storage may be obtained at the end of the rains.

The further great advantage of such a form of weir is that it is perfectly automatic in its action. So much has the value of this property impressed many engineers that they consider any form of non-automatic weir unsound in principle, although the working of most engineering schemes requires human agency for their management.

The great objections to a solid and level weir are that it tends to keep the reservoir full during that part of the year when it is most difficult to effect repairs; it prevents the reservoir from being lowered rapidly when necessary in the case of an accident to the dam; it impounds the earliest monsoon floods, which are always the most heavily charged with silt; and, by maintaining the largest reservoir capacities, it gives all floods passing through the tank the maximum time in which to deposit their silt (*vide* Chapter I., paragraph 36). Further, with it the flood-absorptive capacity of the reservoir cannot be brought fully into play, and hence the length of the weir may have to be doubled beyond what is necessary when that capacity can be properly utilised. Finally, the storage capacity between full-supply

and high-flood levels cannot, as a rule, be made safely available with this form of weir.

183. General Description of the "Stepped Waste-Weir."—To meet the objections to a solid weir with a permanent crest, the "stepped waste-weir" design is put forward. A form adapted to a saddle site is illustrated in Plate 4, and one suitable for a flank site is sketched in Fig. 31. In this latter case it will be best to place the under-sluices and automatic gates some distance away from the end of the dam, so as to avoid the formation of a deep channel at

FIG. 31.



this side, although this position may entail an increase of excavation in the tail channel.

Such a weir may consist of five sections :—

(*a*) A drowned channel or weir ;

(*b*) A clear overfall weir ;

(*c*) An under-sluice section ;

(*d*) An automatic gate section ; and

(*e*) A temporary weir crest.

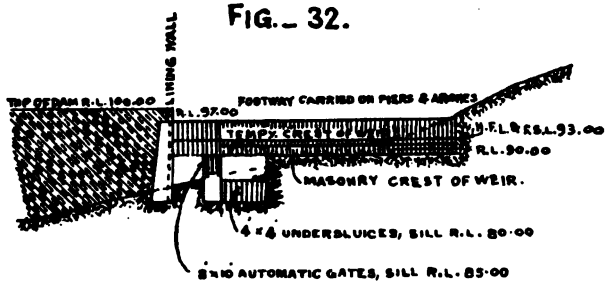
The levels at which the discharge will be passed through, or over, the work will thus vary considerably, and hence the name "stepped" given to this form of weir. By this variation of level, conformity with the natural profile of the ground can be best attained, for it will be but seldom that, for the long length required for a weir, the ground will be level throughout ; hence, with this kind of weir there

may be a considerably lessened amount of excavation in the tail channel.

The stepped weir is best adapted to tanks with certain replenishments, for with them it is not necessary to arrange for securing the complete storage until near the close of the monsoon. It can, however, be used with tanks having uncertain replenishments, but for these it will be advisable to have a greater number of under-sluices placed with their sills at the lowest level practicable and having a maximum discharging power equal to that of the weir crest proper (*vide* Fig. 32).

By this arrangement, on the occasion of a sudden flood,

FIG. 32.



a very considerable lowering of the reservoir surface can be effected. In this case that surface may have to be maintained at a higher level in the early part of the monsoon than will be necessary in that of a tank with a certain replenishment.

189. Advantages of the "Stepped Waste-Weir."—This form of weir meets the objections enumerated in the case of the level weir with a solid crest (*vide* paragraph 187 above), and has the following advantages over that type.

(*a*) It enables the level of the reservoir to be "restricted," or kept low, during the early part of the monsoon, when the rainfall is most continuous and repairs can be effected only with difficulty;

(*b*) It allows the reservoir surface to be rapidly lowered when this becomes necessary in the case of an accident to the dam, (a most useful advantage);

(*c*) It permits the earliest monsoon floods, which are those most heavily charged with silt (*vide* Chapter I., paragraph 36), to be passed out of the tank in the shortest possible time and with the minimum deposit of silt;

(*d*) By keeping the storage of the reservoir as small as possible during the earlier part of the rains, it enables all floods during this period to pass out of the tank quickly and before they deposit all their suspended silt;

(*e*) It permits larger catchments to be utilised without fear of excessive silting;

(*f*) It brings the flood-absorptive capacity of the reservoir fully into play, and allows the capacity between full-supply and high-flood level to be stored for irrigation at the close of the monsoon;

(*g*) It enables the length of the weir to be reduced greatly, (a matter of the greatest importance in the case of a site with a restricted length);

(*h*) It allows a great variety of sites to be adopted for the waste-weir, as it is better suited to the usual natural profile of the ground;

(*j*) It converts a short flood of maximum intensity into a prolonged one of smaller amount, which will less injuriously affect the tail channel;

(*k*) It enables the tail flood to be directed along a defined and comparatively narrow channel, and thus may avoid the cost of lengthy protective works;

(*l*) The deepest part of the weir may be utilised as an outlet for a reservoir with a comparatively small depth of utilisable storage.

It may be added that the stepped waste-weir is adapted only to countries where the rainfall is confined to certain

periods of the year and where storms of great intensity have to be dealt with.

190. Objections to the "Stepped Waste-Weir."—The objections to the stepped waste-weir are two :—

(*a*) It is not automatic;

(*b*) It may not be possible to utilise it owing to the uncertainty of securing full storage at the end of the monsoon.

In respect to (*a*) it may be said that throughout the early part of the monsoon, when the under-sluices and temporary weir crest would be fully open and the automatic gates arranged to open at once, the weir would be automatic, *i.e.*, it would pass the maximum flood possible without the need of any regulation by human agency. It is only at the close of the rains, when the final amount of storage is being, or has been, obtained by the closure of the under-sluices and by the erection of the temporary crest, that the weir will not be automatic. The reservoir surface will, anyhow, fall from high-flood level to full-supply level early in the cold weather, (during which season storms of any intensity are hardly known in India), and it will thus be for only a few weeks at the end of the monsoon that it will be necessary to ensure that the weir is intelligently worked. It must be remembered that the rise of a reservoir, even during a heavy flood, is so slow that it will never be necessary to act with extreme promptitude. If a railway, where a matter of less than a minute may mean all the difference between safety and an accident, can be successfully worked throughout the year by human agency, much more easy will it be to regulate the levels of a reservoir by means of a stepped waste-weir during the few weeks when it will not be automatic.

In respect to (*b*), the objection to this form of weir is more reasonable. There will be certainly much difficulty

in the case of a catchment with an uncertain yield to determine for how long the reservoir surface should be kept at the level of the sills of the lowest sluices without endangering the final loss of storage. The maintenance of the tank at this level is, however, chiefly required in the interest of diminishing its rate of silting, and for such storages this may be more or less sacrificed. The principal utility of the stepped waste-weir for such catchments lies in its power of enabling the full-supply level of the reservoir to be raised to high-flood level at the end of the monsoon. This increase of storage can be safely effected by increasing the discharging power of the weir at its lowest levels (*vide* paragraph 188 and Fig. 32 above).

In the case of reservoirs with unfailing replenishments this objection has, however, but little force. Their catchments will almost invariably produce a sufficient amount of run-off at the close of the rains to complete the filling of the tank. The best catchments will be those which furnish a fair weather discharge after the end of the rains of an amount sufficient to complete the high-flood level storage, as thus a large proportion of the contents of the reservoir will consist of water having originally the minimum of silt suspended in it. An example of such a treatment is furnished by the Assuán[1] dam across the Nile.

191. Detailed Description of a "Stepped Waste-Weir."—The stepped waste-weir proposed in the Máládevi tank project is illustrated in Plates 4, 6, 7, and 8, and may be taken as an example of the general form of this type of weir.

At the flanks the weir is designed with a clear overfall crest 1,004 feet long, (with a net length of sluice-openings between the piers of 800 feet), and having a maximum

[1] "Minutes of Proceedings Inst. C.E.," Vol. clii., Paper No. 3361.

flood depth over it of 4 feet. At the centre are twenty 6 feet by 4 feet under-sluices worked by lifting screws and capstans, and protected against injury from floating logs, &c., by gratings formed of iron rails. Next to these sluices are eight 8 feet by 10 feet automatic gates. Above all, and extending for the whole length—1,133 feet—of the weir crest, is an arcade carrying a foot-path and a tramway on which travelling winches can run: these will work the 10 feet by 4 feet teak shutters, which are used temporarily to raise the weir crest and thus to increase the reservoir full-supply level by 4 feet. It may be noted that 4 feet is probably the proper limit of height for such a temporary crest, and that it would be generally better not to make it more than 3 feet in height: in the present case, owing to the short length available for the waste-weir, this could not be done except by a considerable increase of the under-sluices. At the extreme flanks are small embankments connecting the weir with the hill slopes: these have their tops lower than that of the main dam and are of smaller section than its crest, so that they may act as breaching sections (Chapter II., paragraph 75) should any abnormal flood occur.

On the upstream side the high ground is excavated to the level of the masonry crest of the weir. On the downstream side are excavated cross channels, parallel and next to the weir (*vide* paragraph 181 above), so that the surface of their high-flood discharge may be at an inclination of 1 in 100, and of gradually increasing width to enable them to pass off that discharge. To reduce the depth of the foundations of the weir and to protect them, curtain walls are placed at intervals across these channels. The lowest two of these walls, (one at each side of the under-sluice section), form with it and the "water-cushion wall" a water-cushion, enclosed on all four sides by masonry, in which the whole discharge from the weir is collected and

passed down regularly to the tail channel. The sides of the cross channels opposite to the weir are protected from erosion by masonry lining walls, and small pitched flood embankments are carried parallel to the cross channels and to the tail channel, as far as this protection is necessary, to prevent the weir floods from flowing over the adjacent ground. The tail channel has at first a bed fall of 1 in 100, and afterwards one of 1 in 50, until it meets a natural deep watercourse into which the floods are turned. The widths of the tail channel are gradually reduced so as to save excavation, care being taken that no heading-up and consequent retardation of discharge is thereby produced at the weir itself. By these arrangements the floods are tapped at a low level, and are passed down a definite channel to the river.

192. The Under-sluices, Automatic Gates, and Temporary Weir Crest. — (*a*) *Under-sluices.*—The under-sluices are of the ordinary cast-iron pattern, with planed gun-metal faces sliding on cast-iron frames, similarly faced, and between cast-iron guides.

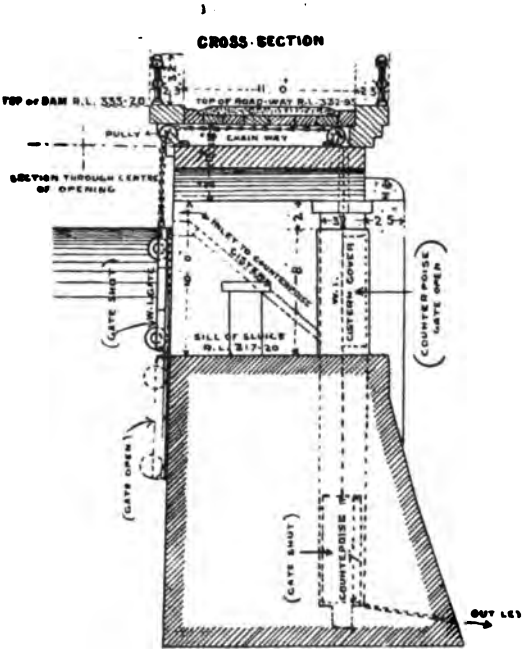
(*b*) *Automatic Gates.*—The automatic gates proposed are similar to those erected at the Bhátgarh reservoir, in the Poona district, which are of the design patented by Mr. E. K. Reinold. These have been tested by actual experience for some years, and have proved quite satisfactory. The principal objection to them is that the gates travel downwards to open, so that they are not adaptable to all situations. Their action may be thus described (*vide* Fig. 33[1]):—

The gate, which may be 10 feet long by 8 feet high, travels downwards by means of rollers from its closed to its open position, and, suspended from it by two chains, one on each side, is a counterpoise weight, which works in a

[1] Buckley's "Irrigation Works in India and Egypt," p. 180; *The Engineer*, Nov. 3rd, 1893, pp. 430, 431.

water-tight chamber, and has a motion reverse from that of the gate. At the calculated high-flood level there is a large inlet pipe leading from the reservoir to this chamber, and at the bottom of the chamber is a small outlet pipe discharging into the air. When the reservoir is at high-flood level the gate is at its maximum height, and then entirely closes the opening between the piers of the sluiceway. Should the reservoir surface rise to a higher level, water will find its way down the large inlet pipe and fill the counterpoise chamber, thus making the counterpoise lose weight, so that it is no longer able to keep the gate in its closed position. The gate then falls, and water is discharged through its sluiceway until the reservoir surface falls below the mouth of the inlet pipe. When this happens the outlet pipe drains the chamber, the counterpoise regains its full weight, and, descending, raises the gate to its closed position. This action of the counterpoise can be called into play by fixing the mouth of the inlet pipe at any designed level, or, by means of a valve worked by hand, admitting water through a pipe to the chamber when desired.

FIG. 33.



Several other forms of automatic gates have been recently patented, but as yet not much practical experience of their working has been gained.

As the automatic gates have to be placed near the crest level of the weir they have not the same rate of discharging power as the under-sluices, which can be fixed at any required lower level. Were a careful watch kept over the reservoir surface when it is at, or near, high-flood level, (and it should be always easy to arrange for this in the case of a work of any importance), automatic gates would not be a necessity, and might be replaced by an additional number of under-sluices. In any circumstances, it will be better to provide for a larger discharging capacity by under-sluices than by automatic gates.

(c) *The Temporary Weir Crest.*—Probably the best arrangement for this will be that shown on Plates 7 and 8, and described in paragraph 191 above. It entails the construction of an arcade on top of the weir, but it is advisable anyhow to go to this expense so as to secure the means of dealing with any part of the weir and so as to gain access to the whole work at any time.

On the large weirs of Northern India there are several ingenious forms of crest shutters, but, owing to the great length of the works, they are of a somewhat cheap description, and it is difficult and takes time to close them, so that these designs do not appear suitable for the crest of waste-weirs which have a comparatively short length and a small crest width. However, such devices up to 6 feet in height are worked, and, if economy is absolutely necessary, they might be used on waste-weir crests, but their height had better be limited to 3 feet. For this, or a smaller, height some form of drum weir should be useful, as it might be designed in sections, say, 200 feet long, so as to be worked by water pressure from the ends of the weir.

193. Flood Calculation of the "Stepped Waste-Weir." —Appendix 14 gives a calculation showing how the stepped waste-weir of the Máládevi project can dispose of an extreme flood by discharging part of it and by allowing the balance to be absorbed by the reservoir. Starting with a discharge of 7,600 cubic feet per second, (which is equal to a run-off of 1·85 inches a day from the catchment, and is a fair small flood), it will be seen that in 13 hours a total flood equivalent to a run-off of 6·96 inches is disposed of. Of this 1·75 inches are passed off by the weir, and the balance, 5·21 inches, is absorbed by the reservoir. When high-flood level is reached, and assuming the reservoir remains at this level, the discharge of the weir crest and of all the sluices and gates will be at the rate of 0·418 inch an hour run-off, thus giving a total run-off during the day of 11·56 inches. These runs-off are far in excess of what may be expected from the catchment, and the waste-weir provision is thus shown to be ample.

The calculated high-flood discharges of the sections of the weir are:—

	Cubic feet per second.
(*a*) Clear overfall weir crest, 800 feet in net length and 4 feet deep	22,827
(*b*) Twenty under-sluices, each 6 feet by 4 feet....	12,000
(*c*) Eight automatic gates, each 8 feet by 10 feet	6,457
Total	41,284

This is equal to a run-off of 0·418 inch an hour from the catchment, and the flood calculation shows that this provision is ample, although a run-off of 1·00 inch per hour would probably have been required had a level waste-weir been adopted.

In addition to the above high-flood discharge, that of the outlet under-sluices (Chapter IV., paragraph 200) is 7,027 cubic feet per second, or at the rate of 0·07 inch an hour run-off from the catchment.

194. Working of the "Stepped Waste-Weir."—The arrangements proposed in the Máládevi tank project may be given as an example of how a stepped waste-weir may be worked in the case of a reservoir with an unfailing catchment.

During the early part of the monsoon the outlet sluices would be kept fully open, and, owing to the flood-absorptive property of the reservoir, it would only be rarely that the waste-weir would come into action. By the beginning of September it would be probably necessary to commence to effect storage above the sill of the under-sluices—R.L. 177·00—at which level the reservoir contents are 2,202·486 million cubic feet, or 0·43 of the high-flood level storage. By the beginning of October the reservoir would be allowed to fill to the masonry crest of the weir—R.L. 196·00—at which level its contents are 4,498·935 million cubic feet, or 0·88 of the high-flood level storage. During October the balance—613·418 million cubic feet, or 0·12 of the high-flood level storage at R.L. 200·00—would have to be stored. These storages represent, respectively, runs-off from the catchment of 6·37, 6·65, and 1·78 inches.

It will be seen from this that up till September the outlet sluices and the waste-weir sluices and crest would be kept quite open and unobstructed, and the reservoir would be thus far safer than one with an ordinary level waste-weir. During September and October the rise in reservoir surface level would have to be carefully regulated. In November the reservoir surface would be maintained at high-flood level, as far as draw-off and replenishment would permit. By the end of that month it would probably

have fallen below it, and the reservoir would thus gain flood-absorptive capacity, which would rapidly increase as the season advanced, until, by the end of December, the masonry weir crest would be probably above water level, and then the temporary crest could be finally removed.

The weir would thus require careful supervision by a superior staff only during September and October, as thereafter in this locality there is no fear of storms. Such a staff would be wanted for the working of the whole irrigation project, and could easily devote its especial attention to the reservoir during this period.

195. Saving Effected by the Adoption of the "Stepped Waste-Weir."—It has been explained in paragraph 166 above that the margin of safety between full-supply level and high-flood level involves what may be considered either as a very expensive, but necessary, addition to the cost of the dam, or as a surrendering of a large amount of storage. This margin is absolutely necessary in the case of a level weir with a solid crest, as it is not generally advisable to attempt to impound much further storage above that crest. Should a sudden and intense flood come down after the temporary crest has been formed on the permanent one, there will not usually be time to remove the upper crest, the flood-absorptive capacity of the reservoir will be considerably diminished by the increased storage effected by that crest, and thus there will be a risk of the failure of the whole work.

The case is different with a stepped waste-weir which has been properly designed with sufficient discharging power below its permanent crest level by means of under-sluices and automatic gates; moreover, the weir arcade will permit of the speedy removal of its temporary crest. With it the storage of the reservoir can be safely allowed to rise at the end of the monsoon to high-flood

level, and thereby the maximum capacity of the tank can be secured without any additional increase to the height of the dam. In the event of a sudden and intense flood coming subsequently, it can be dealt with by opening the under-sluices and automatic gates as soon as it is seen that it is likely to occur, and, thereafter, by removing the temporary crest if this becomes necessary. As the flood subsides, the temporary crest can be refixed and the gates and sluices closed so that the tail of the discharge may be impounded in order to restore the full storage capacity.

As an example of the saving effected by the use of the stepped waste-weir, the estimates in connection with the Máládevi tank project may be given. The original level waste-weir design was estimated at Rs.51,430, but, on account of the deeper foundations subsequently found necessary, it would probably have cost at least Rs.70,000, The stepped waste-weir design was estimated at Rs.156,007, or, say, Rs.86,000 more. The full-supply storages effected by the two forms are :—

	Storage in million cubic feet.
Level waste-weir crest, R.L. 188·00	3,385·602
Stepped waste-weir crest, R.L. 200·00	5,112·353

The latter, therefore, stores 1,726·751 million cubic feet more than does the former, and the value of this, (at Rs.238 per million cubic feet, the rate of the total storage of the project with the stepped waste-weir), is Rs.4,10,967, thus giving a net saving over the level weir of, say, Rs.3,25,000. The two projects compare exactly in respect of the dam, as for each the high-flood level was taken as R.L. 200·00.

Another way of comparing the two forms of weir would be to take them with the same full-supply storage at R.L. 188·00. In regard to the stepped waste-weir design

there would be, in consequence of its reduced height, some saving in the cost of the masonry and some excess in that of the excavation of the tail channel, but, on the whole, there would not be much difference in the cost of the weir itself. The saving in the cost of the dam by the reduction of its top level from R.L. 207·00 to R.L. 195·00 (7 feet above the combined high-flood and full-supply level) would, in this case, amount to about Rs.2,74,000.

A later design for a level waste-weir 1,575 feet long was estimated to cost Rs.1,83,020. Its full-supply level was lower by 2 feet, and its full-supply storage capacity less by 311·557 million cubic feet, than that of the stepped waste-weir design. This diminished storage, at Rs.238 per million cubic feet, is equivalent to a loss of Rs.74,000. Moreover, as the dam has to be raised 7 feet higher to R.L. 214·00 in this case, its cost is increased by Rs.1,95,000. Thus, this level waste-weir virtually costs Rs.2,96,000 more than does the stepped waste-weir.

In the above comparisons only the value of the additional water stored annually by the stepped waste-weir has been considered. It would, however, be fair to take into account the capitalised value of this extra storage, and this would show more truly the economic advantage of that design.

Owing to the largely increased capacity of storage reservoirs at the higher contours beyond that at the lower ones, and to the rapid increase of the quantity of earth-work entailed by the raising of the dam, it will be certainly found in every instance that the adoption of the design of the stepped waste-weir will lead either to greatly increased storage or to a greatly diminished cost of the reservoir.

CHAPTER IV.

THE OUTLET.

I. General Remarks.

196. The Object of the Outlet.—The outlet is the work by means of which the water contained in the reservoir is passed safely through, or round, the dam, so that it may be utilised for the purposes for which the storage has been effected. In the smallest native tanks the outlet is simply a cut through the bank, which is opened when water is required for irrigation, and is closed by embankment when supply is no longer needed. The objections to such cuts are that the discharge cannot be completely controlled during ordinary occasions, and that in the case of high floods or of neglect, breaches are apt to be formed by which the embankment is damaged and storage is lost. In the case of a large work such a simple contrivance is quite out of the question, on account of the depth and of the pressure of the water, while, even for the smallest work, it is desirable to provide proper means of regulation and control.

197. The Location of the Outlet.—The outlet ought to be placed at the head of the most suitable alignment for the distribution works. As, however, the cost of the first section of this alignment is relatively small compared with that of the dam, economy in connection with it is of much less importance than the attainment of the safety of the embankment, and the first consideration should be therefore paid to the proper location of the outlet with respect to the dam itself.

The best position for an outlet through a dam is at the centre of a saddle, or depression in the natural ground, across the centre line of the dam, as then the embankment will settle symmetrically on both sides of it. There will thus not be any tendency to the formation of a crack or slip over it, and any leakage which may occur will naturally find its way at once to the centre of the depression, and will not lubricate the base of the dam.

The worst position for an outlet is on steep, side-long ground, and particularly on the side of the river gorge. The outlet being here placed unsymmetrically with respect to the longitudinal section of the dam, the earthwork on the lower side will have a tendency to move away from it, and any leakage at, or through, the outlet tunnel will lubricate the lower part of the base of the embankment and will increase the tendency to its movement.

When practicable an excellent position for an outlet is near one flank of the dam, or near a cross ridge from its centre line, as then it may be possible to construct the approach bank to the headwall (paragraph 214 below) on natural ground, and not over the centre line of the outlet tunnel (Plate 13, Fig. 4). In this latter position, any failure of the dam through settlement may affect the approach bank.

In order to secure the best foundations, to cross the line of the puddle trench safely, and to prevent any settlement of the dam from affecting the outlet, it is best to have the top of an outlet culvert some depth below ground surface.

These remarks apply to outlets the culverts, or channels, of which cross the dam, and not to tunnels which lie wholly outside the dam (*vide* paragraph 208 below.)

Wherever practicable, the outlet should be on the bank opposite to that on which the waste-weir is placed, so that the floods from the latter will not cross the channel from the former. This point should be taken into account when fixing

the site for the waste-weir, as the location of the channel will be generally settled by the nature of the land to be irrigated. Where, however, both works are on the same bank, the waste-weir floods will have to be passed over the channel in a super-passage, or, under it in an aqueduct, which will entail additional expense and may lead to difficulty.

198. The Number of Outlets.—Outlets at the best are sources of weakness in a dam, and it is therefore desirable to reduce their number as much as possible. In most cases it will not be necessary to have more than one irrigation channel from the reservoir, but, where two or more have to be excavated, either on one or both banks of the impounded stream, the outlets should be as few as possible. Where there are two channels on the same bank, the outlet for the upper one will usually be at such a high level that its construction will not be objectionable. Where the channels are on opposite banks, it should be seen if one outlet cannot be made to serve both by bifurcating the main channel from it at some distance from the dam and by leading one branch across the stream to the other side of the valley, either by means of an embankment, (with a small drainage work under it), or by an aqueduct. In such a case it may be found desirable to lower the sill of the outlet and to place the work nearer to the stream than would be otherwise considered necessary.

199. The Level of the Outlet Sill.—The proper level at which to place the outlet sill will sometimes have to be regulated with respect to that of the land to be irrigated, but, as a general rule, this consideration is not one of the first importance, as the full extent of area to be brought under command can be usually secured by an extension of the irrigation channel. The more important points to remember are that:—

(*a*) Space should be provided below the outlet sill for the

accumulation of silt in the reservoir (*vide* Chapter I., paragraph 42);

(*b*) The capacity of the lower contours of a reservoir being relatively small compared with that of the upper ones, it is generally not worth much extra expense to provide the means of tapping and utilising this part of the storage;

(*c*) The lower the sill, the greater will be the expense and insecurity of the outlet.

200. Subsidiary Uses of the Outlet.—The discharge of an outlet is usually regulated, so that with a small head, of say, 1 or 2 feet, the full supply required for the canal may be given. It is, however, desirable that the discharging capacity should be considerably increased in order to secure the following advantages :—

(*a*) The rapid lowering of the water surface of the reservoir when this is required, either in case of accident to the dam, or for the examination of the outlet sluices;

(*b*) The steadying of the rise of the water surface during the monsoon and the saving of the temporary or permanent waste-weirs by bringing into action, early and safely, the flood-absorptive property of the reservoir (Chapter III., paragraph 184);

(*c*) The diminishing of the silting-up of the bed of the reservoir (Chapter III., paragraph 189 (*c*) and (*d*));

(*d*) Assistance in the closure of the dam (Chapter II., paragraph 139).

Taking as an example the Máládevi tank project, the outlet would have to discharge :—

	Cubic feet per second.
(1) To lower the reservoir 1 foot in a day below R.L. 177·00, the sill level of the under-sluices of the stepped weir (Plate 4, Fig. 2)	1,260

	Cubic feet per second.
(2) To run-off $\frac{1}{100}$th inch per day from the catchment	1,033
(3) To pass 15 inches run-off from the catchment in four months of the monsoon	538

In the case of a reservoir of moderate size, the increased discharge required to serve these purposes can be arranged for by any of the ordinary forms of outlet, but in that of a large tank the only form practicable, on account of the large volumes to be dealt with, is the headwall in the centre line of the dam (paragraph 205 below). An outlet of this type for the Máládevi tank project is illustrated in Plates 9 and 10; this has at high-flood level a discharging capacity of 7,027 cubic feet per second, which is equal to a run-off at the rate of 0·07 inch an hour from the catchment.

In a water-supply reservoir it will be generally necessary to have independent valves for these purposes, so that the increased discharge required may not interfere with the supply to the town (Plates 13 and 14).

II. Different Forms of Outlet.

201. Culvert under the Dam.—This form is illustrated in Plate 13 and Fig. 34 opposite. It is usually the cheapest type, and, when properly constructed, can be made perfectly safe.

The following are objections which have been raised to it and the replies which may be made to them:—

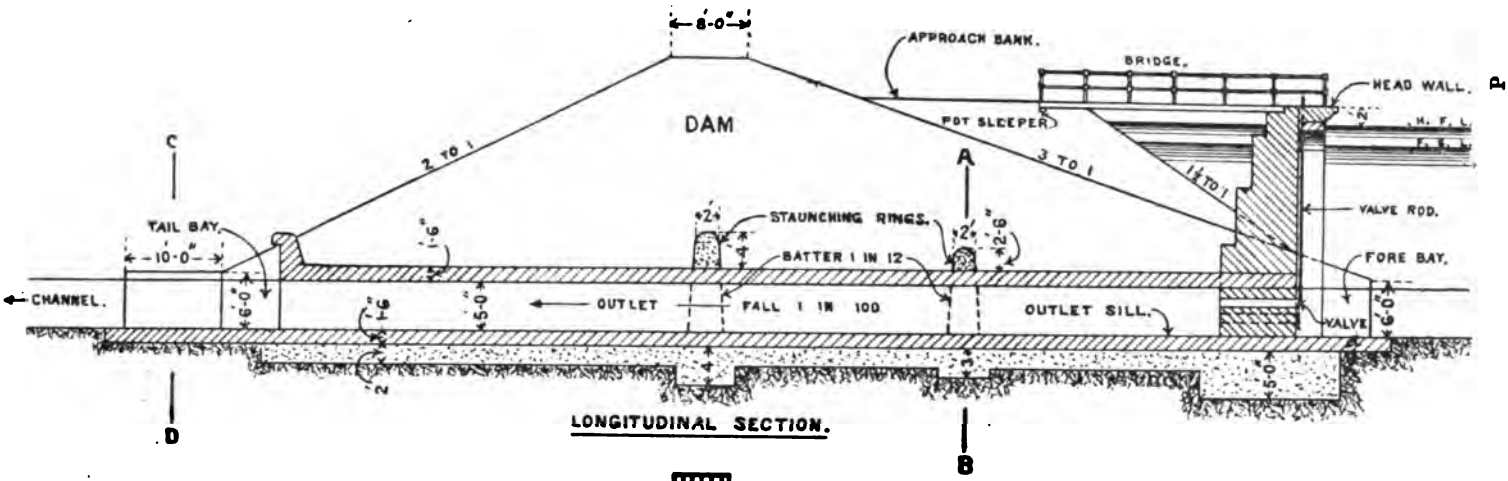
(*a*) It forms a weak point in the dam. This may be admitted in theory, but in practice this objection can be fully met by proper precautions in design and by good workmanship.

(*b*) It cannot be properly inspected or repaired when necessary, as it is under a large mass of made earth. The whole of the construction of the work being executed in

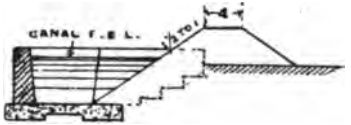
FIG. 34.

OUTLET, HEAD WALL & APPROACH BANK.

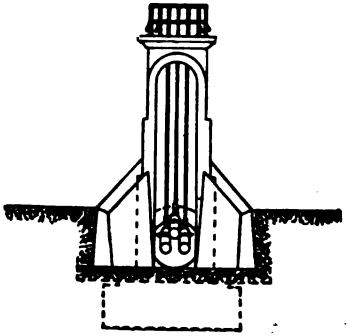
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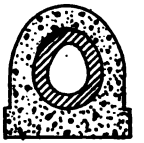
LONGITUDINAL SECTION.



SECTION ON C.D.



END ELEVATION.



SECTION ON A.B.

P

I.R.

the open, there should not be any difficulty in securing first-rate workmanship, and provided a durable material, such as stone, is used, there should not be any need of repairs to the portion next the dam, which is kept at an equable temperature and is not exposed to outside influences. An iron culvert does not appear very desirable owing to the tendency of the material to rust, but, when such a one is constructed, it should be protected by an external ring of concrete and should be built up of rings in segments, each of which may be removed when decayed and replaced by a sound piece.

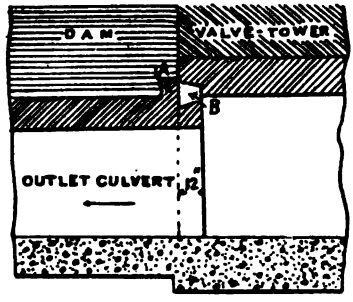
(*c*) It is apt to be disturbed by the spreading out of the dam during settlement. This is rather the fault of the embankment than of the culvert. No such spreading out of the earthwork should occur if it has been carefully executed, and none has occurred in any of the large modern Indian dams. To prevent any possible failure of this kind, it is desirable to build the culvert in a trench and some depth below the surface of the ground, as the friction of the filling against the sides of the excavation will make the culvert practically independent of the dam. In such a trench the length of the culvert will be also reduced to a minimum. Its ends, being well buttressed by the tail- and fore-bays, should tend to prevent any motion outwards.

(*d*) It is liable to be fractured where it crosses the puddle trench. This is the most serious objection to this form of outlet. The puddle trench should never go under the culvert: the latter should have its foundations carried below the bed of the former, but, where this cannot be done, a concrete trench should be there substituted for, and should be well keyed into, the puddle trench. The culvert should have throughout its length a solid, unyielding, homogeneous foundation, and one not liable to be affected

by the percolation of water. This foundation, in all cases practicable, should be of sound rock, but, where this is not available, the culvert should be carried on a wide and deep concrete foundation. No reasonable expense should be spared to secure a thoroughly reliable work, and, if the natural conditions will not permit of this, the site should be rejected.

(*e*) A crack is likely to occur, owing to the unequal settlement of the different heights of the masonry, at the junction of the culvert with the headwall or tower. Such a crack has not been formed at any modern Bombay reservoir, but, should it occur, it would involve only the leakage of water, which would pass harmlessly through the culvert, and would not constitute a danger to the stability of the dam. Fig. 35 shows how the formation of such a crack might be avoided: the outlet culvert is overlapped 12 inches by the concentric arch carrying the superstructure of the tower or headwall. At their junction, A B, the two arch rings are bevelled as shown, and, being disconnected from each other, can settle independently. When the masonry of the tower or headwall has settled finally, which will be before the reservoir fills, the annular wedge space can be made water-tight by running in cement grout.

FIG. 35.



SECTION

(*f*) Leakage is likely to occur between the culvert ring and the dam. Particular care is necessary to avoid this; the junction can be made sound by placing, on the upstream side of the centre line of the dam, a thick

covering of good clay over the culvert ring, and by building this ring with minor projections and with staunching rings, as shown in Fig. 34. These staunching rings should completely surround the culvert and its foundation; they should batter in longitudinal and cross section, and should have rounded tops, so that the earthwork may settle tightly on to them. The ring at the centre line of the dam should have a greater projection than the others, so that it may intercept any flow that may have passed them. The ring which is nearest the reservoir should be situated where it is not likely to be passed by any large amount of infiltration through the dam. Downstream of the centre line of the dam the culvert ring should be cased with dry material, which will act as a drain and will lead away safely any leakage which has penetrated so far without inducing greater percolation from the reservoir.

(*g*) The valve tower, if placed in the reservoir and connected with the dam by a bridge, is in an exposed position, and is liable to injury by ice or to become difficult of access in stormy weather. In India there is no likelihood of ice affecting the works, and the stress of weather there is not sufficient to prevent attention being paid to the valves; these, moreover, can be protected from injury by being surrounded by cages or gratings.

202. General Remarks on Outlet Culverts.

(*a*) *Sound Design and Construction Necessary.*—This form of outlet is the one generally used, and in Bombay no failures have ever occurred in connection with it, as the importance of securing absolutely good design and workmanship has been always realised there. Where the design is inferior and the construction bad, the following opinion doubtless holds good:—"Earthen[1] dams rarely fail from

[1] *The Engineer*, Vol. lxiii., p. 189.

any fault in the artificial earthwork,and seldom from any defect in the natural soil; the latter may leak, but not so as to endanger the dam; in nine-tenths of the cases the dam is breached along the line of the water-outlet passages."

(*b*) *Water-tight Connection with the Trench.*—The trench excavated for the culvert should be considerably wider than the masonry, and should be taken out with slightly sloping sides to give space for a good thickness of puddle, so as to ensure that no leaks are formed along it during settlement. In some cases a concrete casing has been interposed between the ashlar arching and the puddle, or has been allowed entirely to supersede the latter. Every junction of dissimilar materials is, however, likely to produce a leakage plane; this will be the case with concrete and any natural soil, and still more with the smooth surface of the concrete casing and the puddle round it. Puddle will fit much tighter between the rough back of the arching, (the projections of which will prevent the formation of leaks), and the side of the excavation, and, as a greater thickness of it can be used at the same cost as a thinner casing of concrete, it is better to employ it only for surrounding the culvert.

(*c*) *Form of the Culvert Arch Ring.*—Rankine[1] states that the proper form for the line of pressure is the elliptic linear arch, in which the ratio of the half-span to the rise shall not be less than the square root of the ratio of the horizontal to the vertical pressure of the earth. He adds that the entire ellipse may be used as the figure of the arch, or, if necessary, the bottom may consist of a circular segmental inverted arch having a depression of one-eighth of the span.

Another form which has been used is the ovoid (Fig. 34), such as is used for sewers, but in this case the section is

[1] "Civil Engineering," 21st ed., Art. 297, p. 434.

inverted, so as to secure the widest base possible, and this, at the same time, gives the culvert the largest discharging power. For easiness of construction the invert may be replaced by a flat concrete foundation of a depth greater than would contain the true section; in this case the bed of the outlet should be constructed with a central drain to pass off leakage (Plate 14, Fig. 5).

(*d*) *The Construction of the Pavement of the Outlet.*—Where a flat bed is adopted, the pavement should be carefully constructed with large stones breaking joint, the side stones should be well keyed under the arch ring, and all should be set in cement mortar, so as to prevent any displacement by a large rush of water. For the same reason the stones should be built with their longer axes parallel to the flow. The culvert should have a small longitudinal fall, of say 1 in 100, to facilitate drainage and the flow of the discharge.

(*e*) *The Size of the Vent of the Culvert.*—The size of the culvert will primarily be regulated by the requirements of its discharging capacity, but a very small area of aperture will not permit of inspection, and too large a one will tend to make the work fail under the enormous weight of the dam. For these reasons the limiting internal dimensions of an ovoid section may be taken as 40 inches by 60 inches and 72 inches by 90 inches. It is not desirable to have two or more culverts side by side under the dam, as this will weaken it. If more discharging power is required than the single culvert can give, recourse should be had to the type of outlet—the headwall in the centre line of the dam—described in paragraph 205 below.

(*f*) *The Thickness of the Culvert Ring.*—This will depend upon the size of the culvert and the height of the dam over it. The material of the ring should be the soundest stone procurable, and for such stone the thickness

of the voussoirs may vary from 15 inches to 21 inches for dams under 50 feet high, and from 18 inches to 2 feet for dams over 50 feet high.

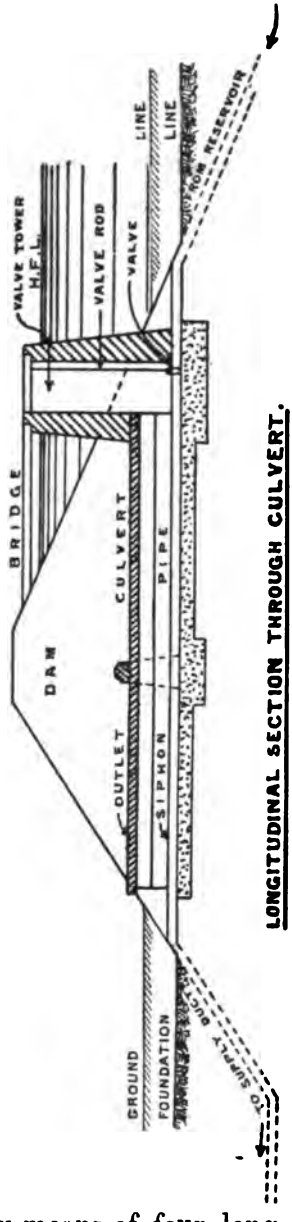
(*g*) *Staunching Rings.*—These have been described in paragraph 201 (*f*) above.

203. Pipe Outlet.—Formerly there were examples of pipe outlets in English dams of large size, but these led to failure, and this form of outlet is now never adopted for large works owing to the dangers attending its use. After the dam has been constructed, the outlet pipes cannot be inspected, they are liable either to break or draw under the weight of the dam, and water may leak through their joints and along them.

There are, however, numerous small native irrigation tanks where this form is the only one economically practicable. In such cases the pipes should be invariably laid on a hard, unyielding bed, should have staunching collars, and should be covered with the most water-tight clay procurable. For the smallest tanks earthenware pipes can be used; in some Madras tanks concrete pipes have been employed; for the larger tanks, with a maximum depth of from 10 feetto 15 feet, iron pipes will be required. These should be laid in a trench and should be packed all round with concrete, which will itself form the water passage if theiron pipe rusts away.

204. Siphon over the Dam.—This form of outlet has been used in England and America, but not in Bombay (*vide* Fig. 36). It is best adapted to small discharges, and is therefore not suited to the large ones required for irrigation, and still less to the maximum ones necessary to secure flood regulation. Moreover, it can be used only where the depth to be drawn off is small—probably 15 feet is the practical limit, but to this can be added the depth below the reservoir surface at which the crest of the siphon can be laid.

The culvert in which the pipes are led through the embankment should have a wide concrete foundation, in order to secure uniform and small settlement, and should be laid in sound natural ground—*i.e.*, not in the heart of the dam. The necessity for having air-tight iron pipes in addition to the culvert, adds to the expense of construction, and will, in most cases, exceed the saving, consequent on their use, of a shorter culvert and lower valve-tower. The design involves a loss of head to produce siphonage, and in many projects this would be a disadvantage.



LONGITUDINAL SECTION THROUGH CULVERT.

FIG. 36.

205. Headwall in the Centre Line of the Dam.—This form of outlet is illustrated in Plate 9, and in Fig. 42, paragraph 215. It may be described as the insertion in an earthen dam of a short length of a masonry one, (the two being united by long staunching walls, one at each side), having a passage for the discharge of the water kept open by means of four long wing walls. The headwall is

pierced by outlet culverts controlled at their upstream face by valves worked by lifting rods and capstans, and these can be made as numerous and of as large a size as if the whole dam were of masonry.

Great care is necessary to prevent the creep of water along the upstream wings and staunching walls, and the latter should therefore be made long and with projecting cross staunching walls as shown in Plate 9. As the staunching walls are only of light section, they are comparatively not costly; they should be battered on all faces and rounded off at top and should have their tops ramped down away from the headwall, so that their length at different heights is adjusted in proportion to the depth and pressure of water they have to withstand. They may be said to act as masonry core walls (Chapter II., paragraph 58) uniting the headwall to the earthen embankment.

The wing walls are shown in Fig. 42 as purely masonry ones, and, when these are adopted, projections and recesses should be formed at their rear faces to unite them with the embankment and thus to prevent the creep of water along them. In Plate 9 the wings are partly of masonry and partly of drystone, for the sake of economy. The drystone should be constructed as described in Chapter II., paragraph 129, and will thus be fairly water-tight and will unite with the earthen embankment.

The foundations of all the walls should be on rock, and should be as good as those required for a masonry dam. The section of the headwall, if it exceeds 50 feet in length between the wings, should be that of a masonry dam of the same height, but, if the wings are closer to each other, they will act as abutments, and the section of the headwall may be somewhat reduced.

This form of outlet should be preferably located in excavation in a saddle, as thereby the length of the wings

can be reduced, and this position will be the best one for the excavation of the temporary waste-weir in which generally it would be located.

206. The Advantages and Disadvantages of the Headwall in the Centre Line of the Dam.—The advantages of this form, permitting as it does of the use of large sluices, are :—

(*a*) The work, being wholly in the open, can be easily constructed, inspected and repaired.

(*b*) The reservoir can be kept low during the early part of the monsoon, when the floods are most silt-laden, and excessive silt deposit can be thus prevented.

(*c*) The reservoir can be lowered rapidly should an accident occur, or should the sluices require examination.

(*d*) As the full storage of irrigation reservoirs is not required for some years after their construction, (*i.e.*, until irrigation extends), this form of outlet will make it feasible to keep the reservoir surface low at first and will thus allow the dam to consolidate under the minimum amount of infiltration. Whenever extra storage is required, (as during a season of scarcity), it can be easily secured by closing the outlet.

(*e*) The outlet can be located at the site of the temporary waste-weir, and, in the case of a high dam, during its construction it can be raised so as to form a higher temporary waste-weir. This will save the necessity for the formation of two or more flood-escapes situated in different places.

(*f*) Sufficient discharging power can be obtained, so as to save the waste-weir from being called into action in all but large floods.

The principal disadvantage of the type is that it will generally be the most expensive form. From Appendix 1 it will be seen, however, that in No. 1, Mukti tank, with a maximum depth of available storage of 41 feet, the

cost of the ordinary culvert outlet was Rs.26,368, while in No. 14, Mhasvád tank, with a maximum available storage depth of 24 feet, but with a much greater discharging power, the cost of the headwall outlet was only Rs.28,392.

The more elaborate design for the Máládevi tank project, illustrated in Plate 9, was estimated at Rs.1,22,372, compared with that of the ordinary culvert type, Rs.21,420, an excess of, say, Rs.1,00,000. This latter was probably a good deal under-estimated: it consisted of a masonry outlet tower 12 feet in diameter, with five 2-feet diameter sluices, a masonry culvert under the dam, and a light iron approach bridge of two spans of 37 feet each.

However, the design for the headwall :—

	Rs.
Includes the cost of the temporary waste-weir channel	10,000
Saves the construction of a 200 feet length of dam, which would cost about	37,000
Saves the construction of another temporary waste-weir channel and the increased amount of embankment over it, which would cost about	23,000
Total	Rs.70,000

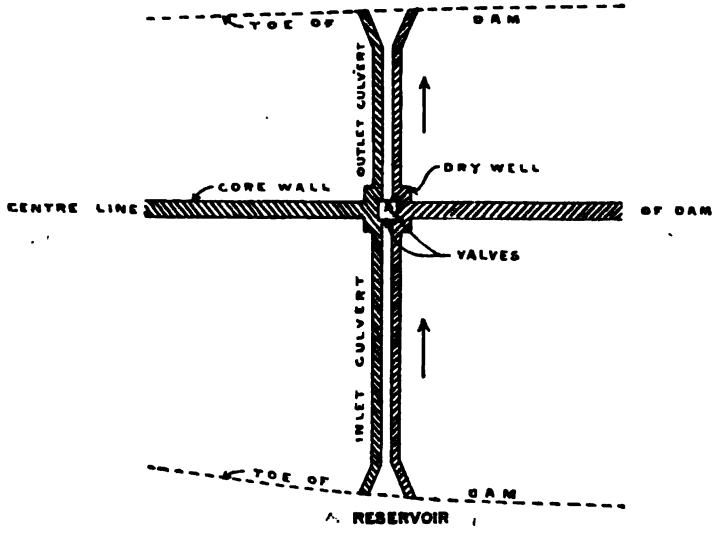
In addition, the headwall secures the advantages described above, the monetary value of one of which, (*b*), the diminution of silting, may be estimated thus:— The average annual yield from the catchment has been estimated at 19,359 million cubic feet, or, say, 15,000 million cubic feet in excess of the full-supply storage of the tank. Assuming, (*vide* Chapter I., paragraph 38), that the volume of the silt is $\frac{2}{3} \times \frac{1}{1000}$ of the volume of the water which contains it, and that the headwall discharge prevents

the deposition of half of this, the amount of silt got rid of annually by it would be :—

$$15{,}000{,}000{,}000 \times \tfrac{2}{3} \times \tfrac{1}{1000} \times \tfrac{1}{2} = 5{,}000{,}000 \text{ c.ft.}$$

Taking the cost of the storage at Rs.238 per million cubic feet, this would amount to an annual saving in loss of storage of Rs.1,190, which, capitalised at 25 years'

FIG. 37.



PLAN OF DRY WELL

purchase, would amount to Rs.29,750 and this can be placed to the credit of the headwall type. These assumptions are all on the side of moderation.

The remaining disadvantage of this type is that it admits water to the heart of the dam, but it should be easy to prevent this by good design and careful construction, as explained in paragraph 205 above.

207. Outlet Tower in the Centre Line of the Dam.[1]

[1] "Minutes of Proceedings Inst. C. E.," Vol. cxxxii., p. 255.

—This is known in America as the "dry well"; it is particularly adapted to the case in which a masonry core wall is used in the construction of the dam (Chapter II., paragraph 58). A culvert is constructed under the dam, and, where it crosses the core wall, a rectangular tower having one, two, or more divisions in plan, is built up as the dam is raised, and in it are placed the valves controlling the discharge (*vide* Fig. 37).

The advantages of this form are :—

(*a*) It saves the cost of an approach bank and bridge;

(*b*) It enables the valves to be easily approached in all weathers;

(*c*) The tower cannot be injured by ice.

The disadvantages are :—

(*d*) Water is admitted into the heart of the dam, but in the case of a work with a masonry core wall this is of small importance, as dependence is chiefly placed on that wall to cut off leakage;

(*e*) The valve tower is liable to be affected by any unequal settlement of the earthwork, which may cause a leak to form and may force the lifting rods out of the vertical. The tower will, however, be the strongest section of the masonry core wall.

With an ordinary dam, without a core wall, these objections would be most serious ones and sufficient to condemn the design for adoption.

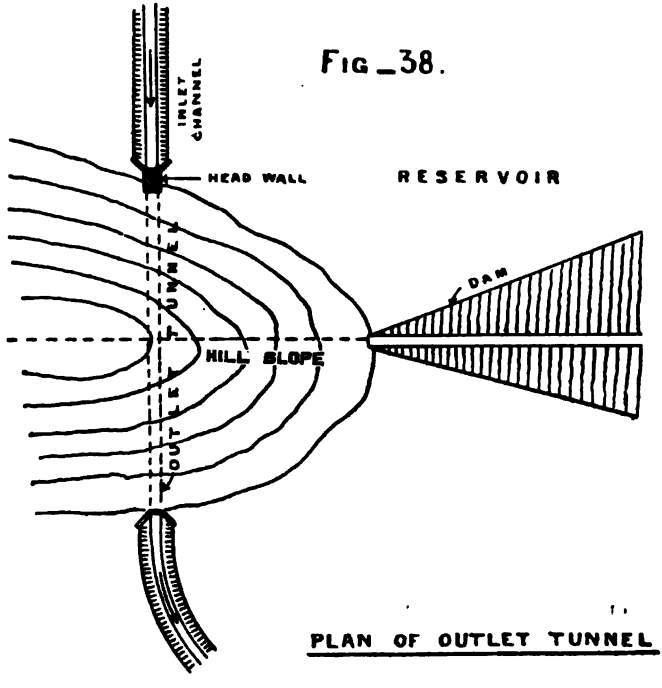
208. Tunnel round the Dam.—This form of outlet, (*vide* Fig. 38), being quite independent of the dam, cannot in any way affect it; it is, therefore, a very safe one to adopt, and is being adopted to a considerable extent in English practice. The objections to its use are :—

(*a*) Its great expense, which is generally not necessary, seeing that other forms of outlet can be made quite safe. In India, moreover, the scale of the natural features of the

country is so large that the cost of the tunnel would be in most cases extremely heavy.

(*b*) The work, being underground, cannot be so carefully supervised as above-ground work. This objection is not a very strong one, seeing that tunnels are well constructed in many situations, and that those for reservoirs will not

FIG. 38.

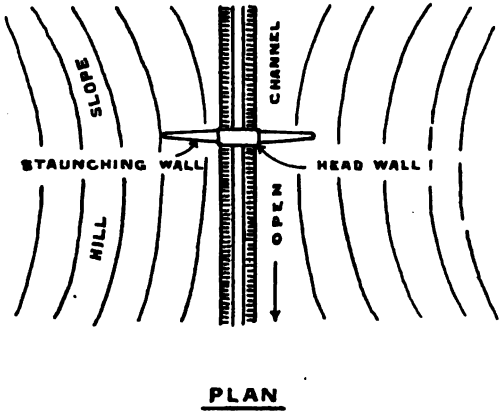


usually be of excessive length nor of great depth below the ground surface.

(*c*) The excavation of the tunnel, which should be in rock, is liable to cause fissures, which may lead to loss of water. This, also, is not a very serious objection, as the fissures will usually not be of any great depth, and the water getting into them will, to a great extent, follow the line of the tunnel and emerge with it.

(*d*) It is difficult to construct the tunnel headwall so as to make it have a water-tight connection with the excavation. This objection can be met by providing the headwall with good staunching walls so as to unite it with the natural ground, and also by constructing staunching rings at intervals. When the rock is sound, it is not necessary to line the tunnel: if the rock is not sound, the expense of lining will usually be so great that it will generally be advisable not to adopt this form of outlet.

FIG. 39.



PLAN

209. Headwall across an Open Channel outside the Dam.—This is of a very simple form: a channel is dug round the flank of the dam, and across it is built a simple headwall, in which are the regulating sluices (*vide* Fig. 39). The excavation of the channel, (unless the spoil from it can be utilized for the construction of the dam), will be expensive and thus this type is adapted only for works having a small full-supply depth above outlet sill. It shares all the advantages of the tunnel form, and, in addition, has that of being easily inspected during construction, and of being easily maintained thereafter. It was adopted in the case of the Medleri tank, No. 20 of Appendix 1; its cost there was Rs.10,409, or nearly one-third that of the dam, which is very high.

210. General Remarks.—(*a*) *The Outlet Channel.*—

The inlet channel upstream of the regulating valves and the outlet channel downstream of the embankment should both be at right angles to the dam for a length at least equal to twice its height, as it is undesirable to have a deep excavation close to its toes. The former should thereafter be led directly towards deep water to shorten its length as much as possible.

(*b*) *Fore- and Tail- Bays.*—On the upstream side the outlet channel should be confined by wing walls from the headwall (or outlet tower); these form the fore-bay. The longer these walls are the more expensive will they be, and the further will the water have access to the heart of the embankment; but the shorter will be the outlet culvert, the nearer will be the headwall to the dam, and the smaller will be the approach bank to it. As a general rule it is not advisable to have the wings very long, and their tops should not be advanced much further than where the dam is 5 feet above ground level.

Similarly, the tail-bay at the downstream end of the outlet should not have its parapet much higher than 5 feet above ground level.

The wings of both bays should extend to where the prolongation of the dam slopes will meet a line 2 feet above the full-supply depth of the outlet, and should be returned into the natural ground, (in a direction parallel to the axis of the dam), for a distance long enough to prevent any slips of the channel excavation from outflanking them.

(*c*) *The Outlet Gauge.*—It is convenient to have a gauge as close as possible to the tail-bay, so that the adjustment of the discharge let out of the reservoir may be effected as quickly as possible. If this discharge is let down to the river to be picked up by the canal headworks, the gauge will conveniently be of the form of a clear overfall weir. If, however, there is no fall available for this purpose, or

if the discharge is at once passed into the canal, the gauge will have to take the form of a gauging run. In any case it is advisable to have a table of discharges made out corresponding to the various depths of the channel, so that the actual quantity let out from the reservoir may be at once ascertained; the gauging run must, of course, be maintained to its correct original section.

(*d*) *Inspection of the Outlet Culvert.*—This inspection should be made twice yearly, once just before and once just after the monsoon, and all damage to the culvert, &c., should be immediately made good. As foul air may accumulate in the culvert, before the inspection takes place a full discharge should be passed down it to clear out such air. Often the culvert can be inspected from the downstream end by reflecting sunlight along it internally by means of a mirror.

III. Regulating Works.

211. General Description.—The regulation of the discharge from the reservoir is effected by means of valves which control the inlet end of the outlet, and are themselves actuated by lifting rods, working up and down the face of the regulating headworks. The headworks will be first described; they may be classified as:—(1) Outlet Towers; (2) Outlet Headwalls; and (3) Dam Slope Outlets.

212. Outlet Towers.—These are invariably used for water-supply storages from which the supply has to be drawn at different levels in order that the clear supernatant water may be obtained continuously as the reservoir surface falls. If the water of the reservoir is excluded, the valves and lifting rods in the interior of the tower can be always inspected. Should, however, the valves for securing double control (paragraph 215 below) be placed on the outside of the tower, they cannot, of course, be inspected

until the reservoir surface falls below them, nor can they be removed and repaired until that happens.

In plan, outlet towers are generally circular, being flattened on the downstream side of the interior to form a seat for the guides of the lifting rods. A tower square in plan is illustrated in Plate 13 ; it has a better architectural appearance than one circular in plan, and is in some ways more convenient than it, but will be generally more expensive.

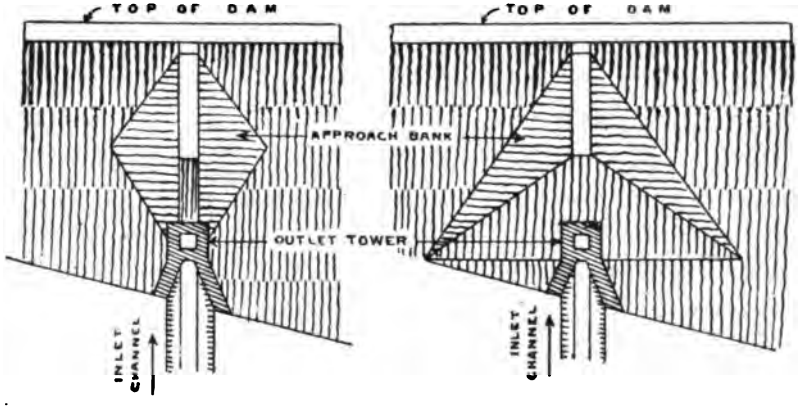
The thickness of the walls of an outlet tower has to be designed with reference to the water pressure they have to withstand at different levels, so that leakage through them may be prevented. A much less thickness is sufficient to secure stability, the tower being subject on all external sides to equal water pressure, and a less thickness will also safely withstand the crushing effect of that pressure. For large towers it will be found economical to construct the walls with masonry facings and a hearting of very fine concrete, (really a coarse mortar), well rammed in between them. To secure water-tightness, the work should be built, or at least pointed, with Portland cement.

Steps for inspection should be provided inside the tower from the top to the bottom, and these had better be arranged ladder-wise rather than spirally, as the latter system does not give a good hand-hold.

213. Outlet Headwalls.—For irrigation storages such refinement is not necessary, as the water is passed out throughout the year from the reservoir at outlet-sill level, and for them a headwall is sufficient. The headwall consists of a simple wall on the reservoir face of which the lifting rods work (*vide* Fig. 34, paragraph 211). The capstans actuating these rods are carried at the top of the wall on a small platform, which is supported by arching thrown across the head of the side pilasters of the wall, or by corbelling from the face of the wall.

214. The Approach Bank.—In order that the outlet headwall should not admit water near to the heart of the dam, it is placed some distance on the reservoir side from the centre line of the embankment. To connect it with the main dam, an approach bank is thrown out from the latter, and from this a small foot-bridge is carried to the headwall (*vide* Fig. 34). The top of the approach bank, the footway of the bridge, and the top of the headwall need not be raised more than 2 feet above high-flood level. The

FIG. 40 FIG. 41



PLANS OF APPROACH BANKS

side-slopes of the approach bank, being of very short length, may be made at an inclination of $1\frac{1}{2}$ to 1, either as sketched in Fig. 40 or in Fig. 41; the latter, having the larger base, is the more secure form, and affords a better cover to the outlet culvert. This bank should have its surface pitched all over except on the top. It is, of course, made at the same time as the main dam, is constructed with it, and is not patched on to it subsequently. Owing to its pyramidal form it is not likely to slip, although its side-slopes are steep. The end of the bridge resting on the

approach bank is most conveniently supported by rails fixed in pot sleepers, which can be packed up to make good any settlement that may occur. Some long approach bridges are supported at their centres by timber or masonry piers, but the better design is to make them with bowstring girders in one span.

The advantage of having the approach bank leading to a subsidiary ridge and not to the main dam has been pointed out in paragraph 197 above. Where this ridge offers good foundations, a masonry foot-bridge can be built from it to the outlet headwork with advantage to permanence and appearance (*vide* Plate 13, Figs. 2 and 4).

215. Double Control over Valves.—If a single valve is placed outside a headwall or on the reservoir side of an outlet tower, it cannot be inspected until the surface of the water falls below it. If, in the case of a tower, a second valve is placed upstream of the first one, (which would then be inside the tower), it is said to give double control to the outlet pipe to which they are fixed in common. By shutting off the supply to the first valve by means of the second one, the former can be examined and repaired, but the latter cannot be attended to until the water level falls below it. Thus, in ordinary designs, the outlet valves of headwalls and the lowest external ones of outlet towers can be examined only in the rare cases when the reservoir is emptied below outlet sill level, and this should never be allowed to occur in water-supply schemes.

The remedy for this defect is a simple one, and consists in constructing, just upstream of the headwall or tower, an inspection chamber by means of a cross-wall built between the wing walls of the fore-bay (*vide* Fig. 42). If necessary, and as shown in that figure, a partition wall may be constructed in this chamber, so that one half of it may be laid dry for examination while the supply continues to

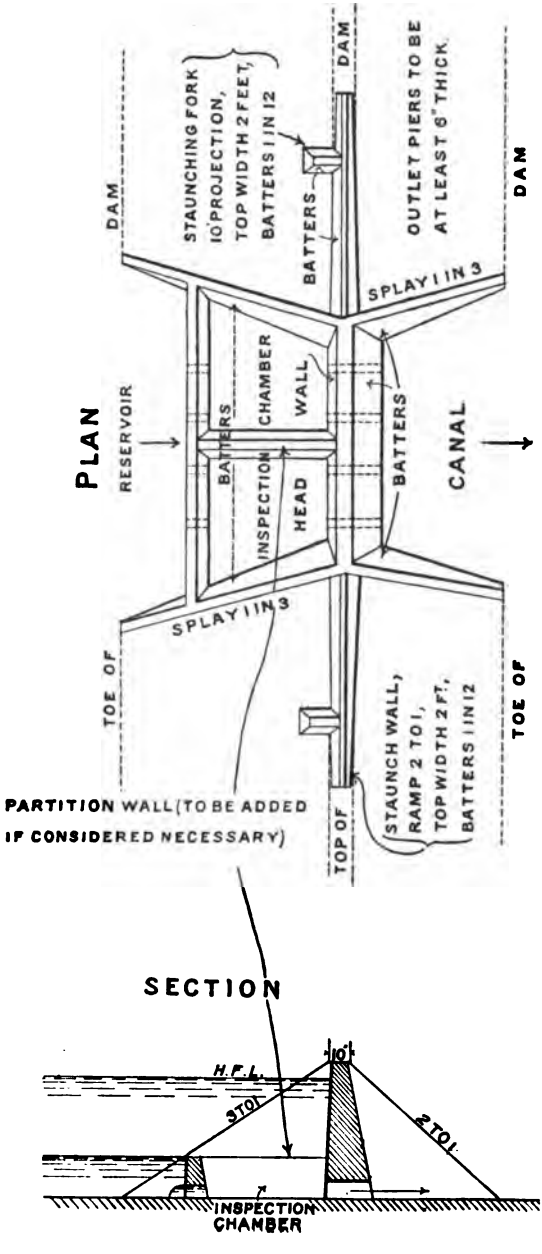


FIG. 42.

HEAD WALL ON THE CENTRE LINE OF THE DAM.

be passed through the other half. In the case of large outlets with large valves, the inspection chamber can be formed by placing wooden needles in front of the grating which is primarily intended to prevent logs, &c., from obstructing the flow or from jamming the sluice (*vide* Plate 10, Fig. 3). For this reason this grating should be fixed in the piers some distance in front of the face of the headwall, and it should be arranged with a slight slope, so that *débris* may float up from it and may not adhere to it.

The cross-wall, (or the top of the needles), should be raised to such a level that at it the storage contents of the reservoir are sufficient to last from the time of inspection, (which would generally be a month before the commencement of the rains), until the first monsoon replenishment may be expected. At the bottom of the cross-wall would be a simple sluice opening, which could be closed either by a simple wooden shutter or by means of sand bags; any leakage through it would pass down the outlet. By this arrangement the lowest valves and sluices can be examined and painted, and, if necessary, repaired each year, while the uppermost ones would be attended to as the water surface fell below them.

216. Dam Slope Outlet.—In this form the expense of a masonry headwall is avoided, but it is applicable only to small reservoirs (*vide* Fig. 43).

The ends of the outlet pipes are turned through a quadrant, so that the valves may be circular ones, and not elliptical, as they would have to be were they placed at the mouths of straight pipes. The valves are situated in small masonry chambers, and the pipes are led into the outlet culvert through a concrete stopping piece. The lifting rods pass through guides fixed to stones, supported on the slope of the dam, and are worked by gearing placed at its top.

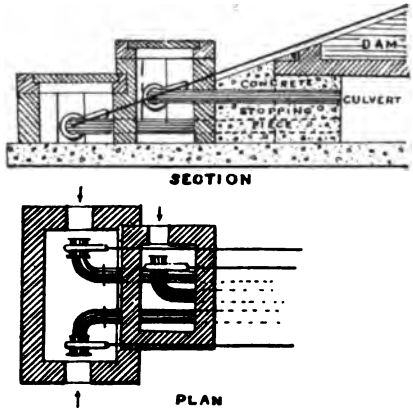
The objections to this class of outlet are:—

(*a*) The settlement of the dam puts the stone supports, and hence the guides, out of line with the proper direction of the lifting rods;

(*b*) The valves cannot be inspected until the reservoir is dry;

(*c*) Great force is required to work the long lifting rods (the lengths of which are that of the slope of the dam and not of its vertical height); these rods rest on numerous guides

FIG. 43.



and, in the usual plan, actuate elliptic-shaped valves of areas increased beyond those of circular ones. The guide friction could, however, be reduced by using rollers, and the valve area by the device above described.

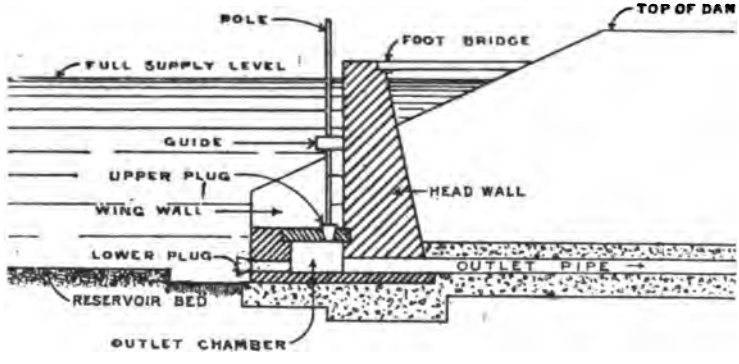
IV. VALVES.

217. Ordinary Valves. — For water-supply purposes, when the rate of supply is small, as it generally is, the commercial pattern of water-valve can be economically used. In this the valve travels up and down a screwed

spindle fixed in a cast-iron casing, and therefore its position cannot be indicated by the lifting rod itself. For medium-sized irrigation works two or more of these may be used in conjunction with each other. For large irrigation works, where large discharges are required, special valves are made. These consist of three parts—the fixed seating, the fixed guides between which the movable gate works, and the gate itself. Examples of these are illustrated in Plate 11 and Plate 12.

All the surfaces in moving contact should be faced with

FIG_44.



PART LONGITUDINAL SECTION

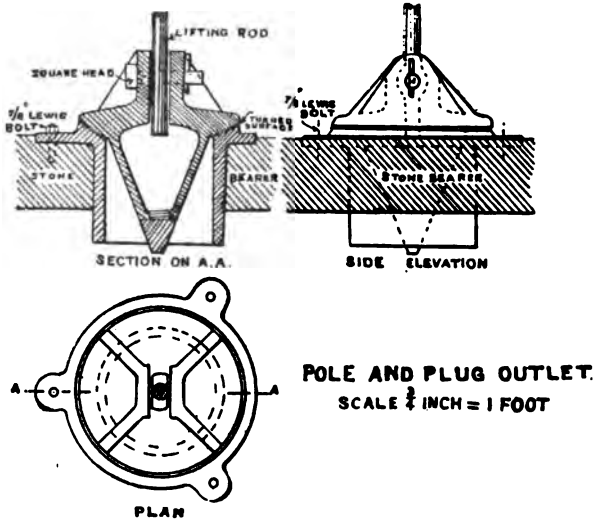
planed brass, or gun-metal, to diminish friction, to prevent rusting, and to make a water-tight joint. As the seating for these large gates has to be made in more than one piece, great care must be taken to fix it to a perfectly true plane. The truth of the seating can be tested by stretching threads diagonally from its opposite corners and by then passing up horizontally another thread, which should throughout its course just touch the diagonal threads and the vertical sides of the seating.

The masonry sill leading to the sluices should be altogether

below the seating, and its upper surface should be sloped downwards to the reservoir to direct the outflow upwards, so as to scour out any silt which otherwise might be deposited at the base of the seating. To prevent the valve from jamming, it is best to make its bottom line part of a circle and to bevel it slightly at the centre, as thus the valve will be guided on to the sill of the seating.

218. "Pole-and-Plug" Valves.—In some small native tanks the outlet head (Fig. 44) consists of a small masonry

FIG. 45.

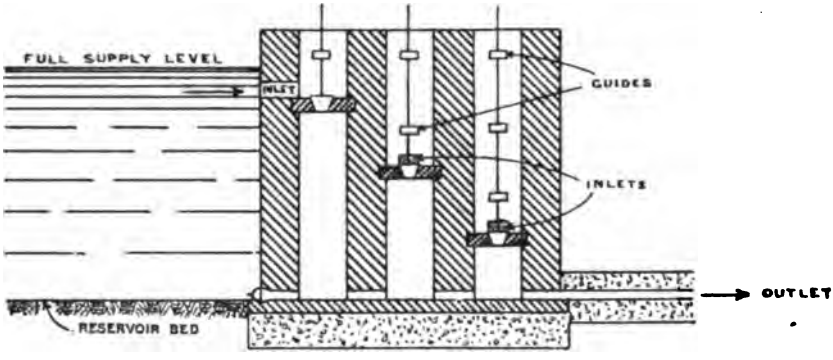


chamber roofed with a slab, which is pierced by an inlet hole the discharge of which is regulated by a conical wooden plug fixed to a pole. A second hole is made horizontally in the front wall of the chamber at outlet-sill level, which, in this case, is the silted-up bed of the reservoir, and this also is closed by a conical plug which is regulated by hand when the water level falls below the roof of the chamber.

This form of valve has been elaborated in iron as sketched in Fig. 45. It is simple, and can be easily worked and

inspected; also, it enables the contents at the bottom of the reservoir to be drawn off with the maximum head.

FIG_46.

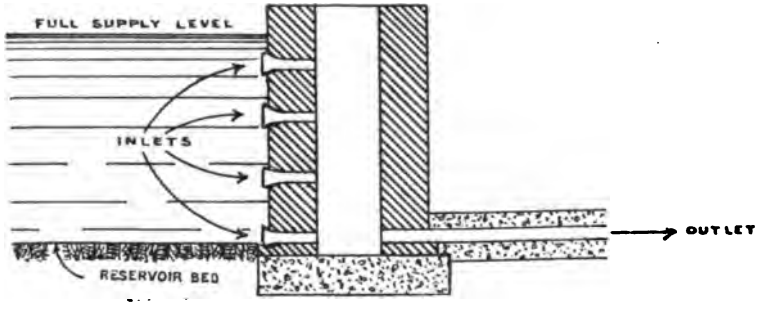


SECTION

Under a great head of water, on account of its form, it would be difficult to work, and would be subject to much vibration.

By using a number of these "pole-and-plug" outlets, at

FIG_47.



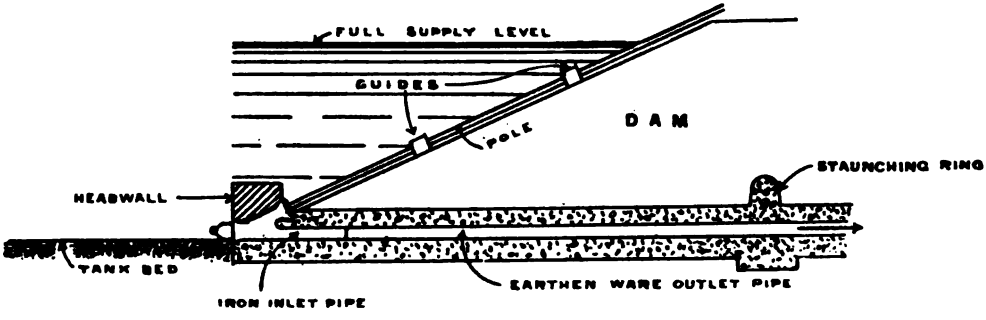
SECTION

different levels in different chambers (*vide* Fig. 46), it is easy to maintain a practically constant discharge by lifting

the different plugs as the water level falls. Or the same may be arranged with simple plugs (*vide* Fig. 47) removable by hand.

Another form of "pole-and-plug" outlet on the principle

FIG. 48.

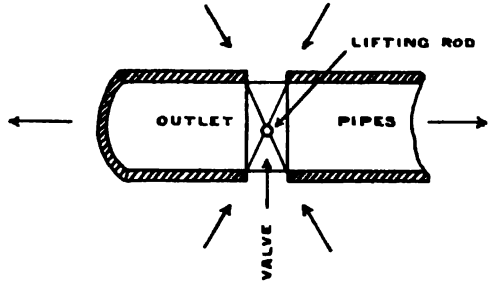


of the "dam slope outlet" is sketched in Fig. 48. This can be used up to depths of 10 feet: it dispenses with the need for a headwall, but, if thought desirable, its upper limb can be made vertical, a headwall can be built and the pole can be made to travel up and down this.

219. Equilibrium Valves. — Working of Ordinary Valves.—The principle of the equilibrium valve is that, as water acts on all sides of it, it is not kept on to its seat by water pressure, and therefore the force required to work it is considerably reduced (*vide* Fig. 49). The valve is slightly tapered so that it fits water-tight on to its seat only when it is fully closed. Such valves are used in waterworks practice, where they frequently have to be worked, but they are not adopted for irrigation schemes, as they are less water-tight, and require more space and therefore larger headworks than do ordinary valves. The expense of having a few extra men for the ordinary sliding gate during the few times it has to be worked is not

great. A better arrangement for utilising the water pressure is to have a turbine by which the capstan heads can be revolved and the lifting rods actuated.

FIG_49.



SECTIONAL PLAN.

220. Testing Gates—Calculations for Gates.—Gates of any considerable size should be tested at the manufacturer's

FIG_50.

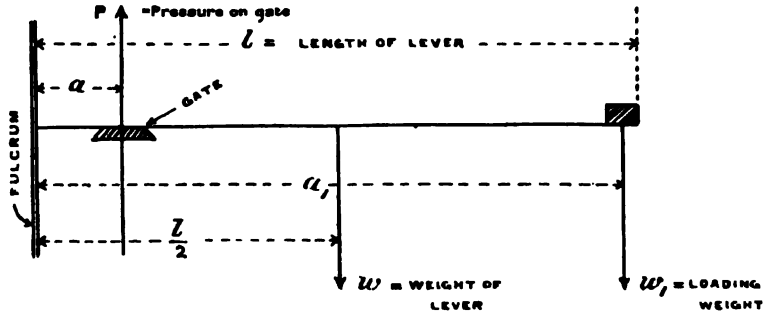


DIAGRAM OF FORCES

yard before delivery on to the works. The gate is laid horizontally so as to bear only on the longitudinal members of the frame, which will be its most unsupported position

in practice, (*i.e.*, when it is not fully closed), and weights are placed on it. A convenient arrangement is to put the weights on a series of levers, (half right-handed and half left-handed), as this obviates the necessity for heavy foundations. Taking an actual case of a gate 5 feet broad and 7½ feet high with its sill 29 feet below high-flood surface, the test, as shown in Fig. 50, was :—

P = pressure on gate
w = weight of lever = 840 lbs.
w_1 = loading weight = 417 lbs.
l = length of lever = 25 feet
a = 3½ feet
a_1 = 24½ feet

Then $P = \frac{wl}{2a} + \frac{w_1 a_1}{a}$

$= \frac{\overset{\text{lbs.}}{840} \times \overset{\text{ft.}}{25}}{7 \text{ ft.}} + \frac{\overset{\text{lbs.}}{417} \times \overset{\text{ft.}}{24\frac{1}{2}}}{3\frac{1}{2} \text{ ft.}}$

$= 3{,}000 \text{ lbs.} + 2{,}919 \text{ lbs.}$

$= 5{,}919 \text{ lbs.} = 52{\cdot}85 \text{ cwts.}$

As six similarly loaded levers were used, the total pressure on the gate was :—

$$6 \times \frac{52{\cdot}85}{20} \text{ tons} = 15{\cdot}85 \text{ tons on the centre line of,}$$

and = 31·71 tons evenly distributed, over the gate.

Under the action of the test the gate should not be deflected appreciably. The amount of deflection would be ascertained by stretching a thread across the gate, and by measuring its distance from a fixed point when the gate was loaded and when it was unloaded.

The gate area = 5 feet × 7½ feet = 37·5 square feet.

The depth of the centre of gravity of the gate below the water surface $= 29 - \frac{7\frac{1}{2}}{2} = 25{\cdot}25$ feet

The water pressure on the gate $= \overset{\text{Sq. feet.}}{37{\cdot}5} \times \overset{\text{Feet head.}}{25{\cdot}25} \times \overset{\text{Weight of water per cft.}}{62{\cdot}43}$

$= 59{,}113 \text{ lbs.} = 26{\cdot}39 \text{ tons.}$

The force required to lift the gate is :—

	Water pressure. Tons.	Coefficient of friction.	Tons.
The frictional resistance of the gate on the guides	= 26·39	× 0·3	= 7·917
The weight of the gate and the rod....			= 1·528
Allowance for extras			= 0·555
Total force required			= 10·000 tons.

The compressive strain which can easily be put on the lifting rod by eight men is :—

$$\underset{\text{Men.}}{8} \times \underset{\text{per man.}}{20} \times (\underset{\text{Circumference of capstan path.}}{2 \times 3{\cdot}14 \times 3{\cdot}5 \text{ feet}}) \div \underset{\text{Pitch of screw.}}{\tfrac{3}{4}\text{ feet}} = \underset{\text{Lbs.}}{5{,}626{\cdot}8}$$

$$= \underset{\text{Tons.}}{25{\cdot}12}$$

This number of men should therefore suffice to lift the gate.

221. Suggested Arrangements for Gates.—The ordinary form of gate involves the construction of some kind of headwall, or tower, up the face of which the lifting rod works —in most cases under water, so that it cannot be inspected. If, instead of this, the valve were worked by bevel gearing placed at the downstream end of the outlet culvert, and the rods were passed through the culvert, the headwall would be saved, and the lifting rods could be inspected at any time. In this case the valve could be given either a horizontal or a vertical motion; the end of the valve rod would be stepped into the valve seating, and would have a pinion engaging with rackwork fixed to the valve.

To reduce the force required to overcome the weight of the valve and the rods, counterpoises might be used, or the gates might be coupled in pairs to balance each other, one falling and the other rising to open the sluices, and *vice versâ*.

222. General Remarks.—(*a*) *The Outlet as a Flood-Regulator, &c.*—As explained in paragraph 200 above, the outlet can be used as a flood-regulator, and also to lower the tank level rapidly. For these purposes the valves should be made larger than is necessary only for the supply of irrigation water, or additional valves should be provided.

(*b*) *The Outlet as a Source of Water Power.*—In paragraph 219 above it has been stated that the outlet sluices might be raised by means of a turbine worked by the pressure of the water in the reservoir. The pressure of the reservoir also might be utilised to furnish water power for industrial concerns, especially when the discharge is returned to the river and is not at once sent down the canal, as, in that case, there will be greater head available. The water thus made use of will be afterwards available for irrigation. Unfortunately, owing to the isolated position of most reservoirs, manufacturing operations cannot usually be economically conducted near them, but with the advent of electrical installations this difficulty may be overcome. Another objection to making thus a further use of the stored water is that its discharge will have to be regulated solely by the requirements of irrigation, and may therefore be too irregular for being utilised for a manufacturing operation. This might be remedied by having an auxiliary steam plant for use as a stand-by on occasions when the water power was not available. Seeing that the country must develop, and that scientific applications must improve, it is desirable, when constructing an outlet, to provide means for supplying water power, as this cannot be done subsequently without greatly increased expense and difficulty.

223. Lifting Rods and their Adjuncts.—(*a*) *Lifting Rods.*—The lifting rod should be attached to the gate below its centre of gravity. For the largest form of gate it should pass through a cored pillar, forming part

of the casting of the gate, and should be secured by a screwed nut below it (Plate 11, Figs. 1 and 2). For smaller gates the end of the lifting rod should be forged out into two straps bolted to the gate, and extending diagonally nearly to its edges at the bottom.[1] The object of these arrangements is to prevent the gate from getting during its travel a sideways motion, which would tend to make it jam in its seat.

In one form of rod (*vide* Plate 11, Fig. 2) its top has a screw thread cut on it which engages with the female screw of the capstan head, through which it passes and by which it is actuated. The rod for the length which passes through the top (square) guide is made square in section, so as to be held by it, and is thus prevented from rotating with the capstan head. With this form of gear the whole lifting rod works up and down, and its top is always visible above the capstan head, so that the position of the valve connected with it can be readily ascertained.

In another form (Plate 12, Figs. 1 and 2) the top of the lifting rod proper is bolted to a cast-iron pillar having a female screw cut in it, into which the screw from the capstan head works, and, by revolving, causes the pillar and the lifting rod to move up and down. The pillar is square in section externally, and passes through a special square guide which prevents its rotation. The weight of the pillar adds to the weight to be lifted, and, as the screwed lifting rod does not travel through the capstan head, its motion and the position of the gate cannot be directly seen. To determine that position it is necessary to bolt on to the top of the cored pillar a "tell-tale" rod, which will always project above the top of the head-wall and will move up and down as the gate rises and falls.

[1] "Minutes of Proceedings Inst. C. E.," Vol. lxxvi., Plate 2, Fig. 10.

(*b*) *Guides.*—To prevent the lifting rod from buckling, it is made to pass at intervals through guides or plummer blocks. The lower guides (Plate 12, Fig. 5) have circular holes, but, as explained above, the uppermost one (Plate 12, Fig. 7) has a square one to prevent the rod from rotating. Care should be taken in spacing the guides that they will not interfere with the travel of the joints of the lifting rod, as these cannot pass through them.

(*c*) *Joints.*—The valve rods are liable to buckle, as an extreme amount of thrust comes upon them when the valves jam or do not work freely from any cause. It is therefore advisable to make them of mild steel and of somewhat fuller dimensions than theory requires. In particular the joints are apt to give: for rods up to 2 inches in diameter the ends of the lengths should be forged out flat and formed into lap joints, secured by bolts passing through them; for rods of larger diameter their ends should be scarfed, and the joint should be confined in a strong collar and secured by slightly tapered bolts passing accurately through them and it (Plate 11, Fig. 16).

(*d*) *Stops.*—The positions of the gate when fully open or fully shut should be fixed by stops, and arrangements should be made to indicate visibly at the capstan when the gate has reached them so that the working of the capstan head may be then stopped.

224. Capstan Heads.—Plate 11, Fig. 4, illustrates a form of capstan head through which the head of the lifting rod rises and falls, and Plate 12, Fig. 1, one through which it does not move vertically. In each instance the capstan is bolted down to its foundation and its internal brass lining is free to rotate. In the former, the screwed head of the lifting rod works up and down through the internal brass, which is cut into a female screw. In the

latter, the lifting screw has annular collars on it, which engage in recesses in the brass lining and prevent it from rising. Below the capstan head it is cut into a screw which engages with a female screw in the head of the hollow pillar forming the top of the lifting rod proper, so that, as the lifting screw revolves, the hollow pillar is drawn up or forced down.

In Plate 12, Figs. 8, 9 and 10, is shown a better design, in which the lifting screw works against a bed step and is separate from the lifting rod; the latter is pinned to a long link carried by a movable stud engaging with the screw. Any deviation of the lifting rod from the vertical is thus not communicated to the screw, and grinding action is thereby avoided.

CHAPTER V.

MISCELLANEOUS.

I. Water-Supply Schemes.

225. Proximity of Storage to the Town Necessary.—Owing to the expense of conveying water in a sanitary duct from the storage reservoir to the population to be served, it is necessary in India, in the case of all but very large towns, to select the site for that reservoir within a few miles of the town which it has to supply. The choice of site being thus restricted to a small area, the probability is that an economical one will not be available, and that the cost of storage will therefore be high. This is not a matter of so much importance in the case of a water-supply scheme as it would be in that of an irrigation project, from which a remunerative return is expected. Although, of course, economy is desirable for the former, the quantity of storage required for it will be comparatively small, and therefore the proportion which its cost will bear to that of the whole scheme will be generally less than it will be in the case of the latter.

226. Amount of Storage, &c., Required to Ensure Certainty of Supply.—The amount of storage required for a water-supply scheme is much less than is necessary for irrigation. Taking the daily consumption per head as 12½ gallons, and making an allowance for loss by evaporation as in Appendix 7, one million cubic feet of storage will suffice for one year for the requirements of 1,000 persons, an amount which is calculated in that Appendix to be sufficient for the irrigation of not more than four acres. On

the other hand, as it is imperative that the water-supply for a town should never fail, it is necessary that for it there should be a storage sufficient to last for two years.

To secure this certainty, the catchment area should be ample, and, if the natural one is not sufficient, the run-off from subsidiary catchments (Chapter I., paragraph 9) should be impounded in subsidiary reservoirs or diverted into it by catchwater drains. An additional advantage of a subsidiary storage reservoir is that if it is large enough, and if the main one can be arranged to feed it, as well as to be fed by it, it will, when necessary, be feasible to use either of them exclusively in parts of alternate years. The one then not in use can be drained and its bed ploughed up and allowed to desiccate, which will improve its sanitary condition.

In all ordinary situations the reservoir can be formed by means of an earthen dam, but, should there be exceptional circumstances rendering this in the least degree a risky form of construction, a masonry dam should be built instead, so as to gain perfect security of supply.

227. Sanitation of the Catchment Area.—Conservancy of Villages in the Catchment Area.—Filtration.— In respect to the amount of the yield of the catchment, there is no difference between the physical conditions which should obtain in the case of a water-supply reservoir and in that of an irrigation one (Chapter I., paragraph 7). In the former there is, however, the additional necessity for securing a clean, sanitary drainage area. The only absolutely safe gathering ground would be one consisting entirely of uninhabited, uncultivated, unpastured, and barren sheet rock of an insoluble nature. Such can be seldom found, and it is therefore necessary in a good water-supply scheme dependent upon a storage reservoir, to have recourse to filtration to remove the defects due to the nature of the drainage area. It is, however, most

desirable to make the catchment as sanitarily perfect as possible. There is, of course, pollution from manured lands, but the amount of unburnt manure used in India is usually small, and the fact that it is ploughed into the ground will still further reduce its potency for evil. The most harmful thing in a catchment is a collection of human inhabitants with their animals, and the pollution from these increases with their number and their nearness to the reservoir or to its tributary streams.

Probably the simplest effectual way of dealing with a village in a catchment area would be to surround it and the places reserved for defæcation by an embankment or by catchwater drains. The drainage from this area would then be led to a small tank with a storage capacity equal to at least one-fourth of the maximum annual yield of its catchment and this storage could be utilised for the irrigation of fields, also embanked around, the tank being emptied four times during the monsoon. The embankments round the village and the area to be thus irrigated and the dam of the small tank would all have puddle trenches under them, and would be constructed perfectly staunch. During the fair weather the bed of that tank and the lands irrigated from it would be ploughed up and allowed to desiccate. This irrigated area should be acquired, and might be let out on liberal annual leases, coupled with the condition that the lessees would be held responsible for the proper working of the system. Such a treatment would interfere but little with the habits of the people, but would, of course, involve careful supervision.

During the construction of the main and subsidiary storage reservoirs for the water-supply scheme, the work-people should be camped below the lowest dam, and should not be allowed to resort upstream from it for natural purposes.

Where filtration is adopted, it will be generally best to place the filters close to the storage reservoir, so that only filtered water may enter the pipes and the spread of pathogenic germs in them may be prevented. Moreover, in this situation probably the ground levels will be more suitable for their location, and there will be a larger area with cleaner surroundings than will be possible for them near the town itself. In schemes having open ducts from the reservoirs, the filters must, perforce, be placed near the town, as these ducts are themselves liable to contamination.

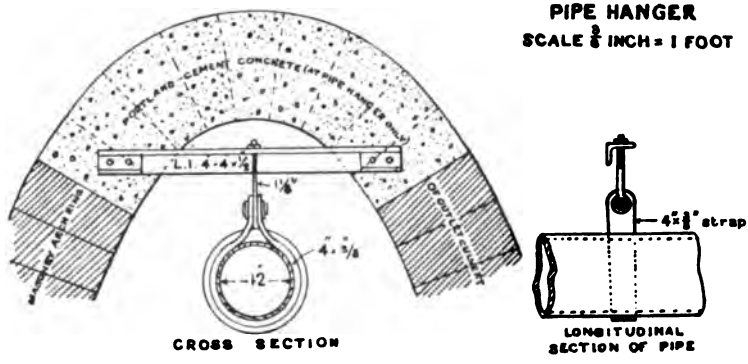
228. The Works of a Water-Supply Storage Reservoir.—In respect of the dam and waste-weir, the design for the storage reservoir for a water-supply scheme will be the same as that for an irrigation project.

For the outlet it will be necessary to arrange to draw off the water from about 2 feet below the surface of the reservoir continuously as its level falls, and to have double control of the valves (Chapter IV., paragraph 215). In Plates 13 and 14 is illustrated an outlet for a water-supply storage reservoir. The water is drawn from the reservoir through a series of inlet pipes at different levels, each of which is controlled by a valve in the interior of the water tower, and is protected externally by a fine grating in a bell-mouth. The inlet pipes all communicate with a central stand-pipe, either directly or by means of short branch pipes at right angles to the inlet pipes, and these branches carry the guides for the lifting rods of the lower valves. The individual lengths of the pipes are designed so as to be of as few patterns as possible. Downstream of the stand-pipe is a valve at the head of the outlet pipe, which thus gives double control to all the inlet pipes. Upstream of the tower is an inspection chamber, which provides treble control for the two lowermost valves. At the base of the tower are two large unwatering pipes, to be used when

the tank level has to be rapidly lowered (Chapter IV., paragraph 200). All the valves are thus inside the tower, and can be inspected at any time and can be taken out and repaired when necessary, the water then being shut off from them by placing a tarpaulin, &c., over the inlet grating.

The outlet pipe is shown suspended from the crown of the culvert arch in Fig. 51, so that its level can be adjusted by means of the suspending bolt. This is a convenient position for the main, as here it does not interfere with

Fig. 51.



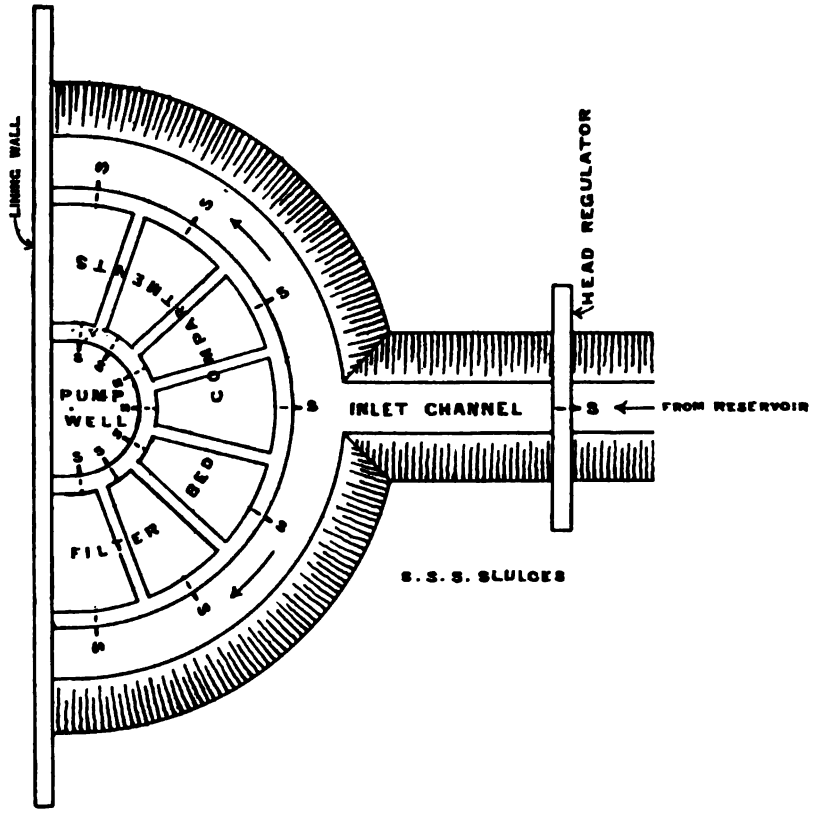
the drainage of the culvert nor obstruct the discharge of the unwatering valves, while the supply main itself can be easily inspected.

When the storage reservoir is below the level of the town, the supply for the latter will have to be pumped up from the former, and an outlet may thus become unnecessary, which is an advantage (Chapter IV., paragraph 198).

An arrangement for combining the inlet to the pumps with a filter is sketched in Fig. 52. The inlet channel is excavated till it reaches ground above the high-flood level of the reservoir, and across it is built the inlet head

regulator and the filter chamber. The latter is designed with a series of independent compartments, each of which can be shut off from the others, its sand washed, and its level

FIG_52.



PLAN OF INLET FILTER

adjusted to suit that of the reservoir as it falls, so that the operations may, if necessary, be made continuous. Direct communication with the reservoir can be shut off by the headwall across the inlet channel; if the subsoil is suitable for the purpose, the water can be allowed to percolate

through it to the filter, which will greatly aid in its purification.

This design was made before modern investigations showed that the efficiency of a filter depends chiefly upon the bacterial scum which forms on the sand, and it might not be approved now on account of the small size of the compartments. However, these could be increased greatly beyond what is shown in the diagram without much cost, as only the pump well wall would require to be made of a section to withstand the pressure of the full head of water in the reservoir.

229. Utility of Water-Supply Storage Reservoirs as Famine Relief Works.—Owing to their situation near a large town, water-supply storage reservoirs form useful works for the employment of relief labourers, who thus can be easily supervised. The works, being comparatively small, can be more nearly completed during a single season of scarcity than larger ones, and will be of greater public utility than an irrigation scheme which directly benefits a smaller number of the population. Even if the municipality concerned subsequently refuses to pay for the full value of the work done, the loss to Government may not be so great as if an unremunerative irrigation scheme had been undertaken instead.

II. Arrangement of Works.

230. General Arrangements.—Detailed notes of general arrangements are given in Appendix 22. Before commencing the actual execution of work, great care should be taken to arrange everything so that due progress may be made uninterruptedly throughout its construction. The workshop, stores and office should be conveniently placed with respect to the work as a whole, and, for the sake of security and quiet, should be removed as far as possible from

existing habitations. The camps for labourers should be carefully selected with respect to sanitary conditions, a bazaar should be opened under supervision, and all should be placed under the charge of a medical officer. The camps should be set out in regular lines; each camp should be well separated from its neighbours, and should, as a rule, not contain more than 200 huts or 1,000 people. The sites should be carefully selected on high ground, removed from undergrowth, and not too close to the working area. The huts should be on raised plinths 1 foot high, with main roads 10 yards wide and cross roads 5 yards wide. The huts may be of bamboo matting, grass, &c., as these generally can be constructed by the people with a little assistance in the shape of material. A hospital should be built near the quarters of the medical officer, and so as not to be a danger to the camps while, at the same time, not being far removed from them. The greatest precautions must be continually taken to conserve the drinking-water supply, and to keep the working area and camps as clean as possible, for which purpose conservancy and sanitary guards should be placed under the medical officer (*vide* Appendix 22, paragraph 29).

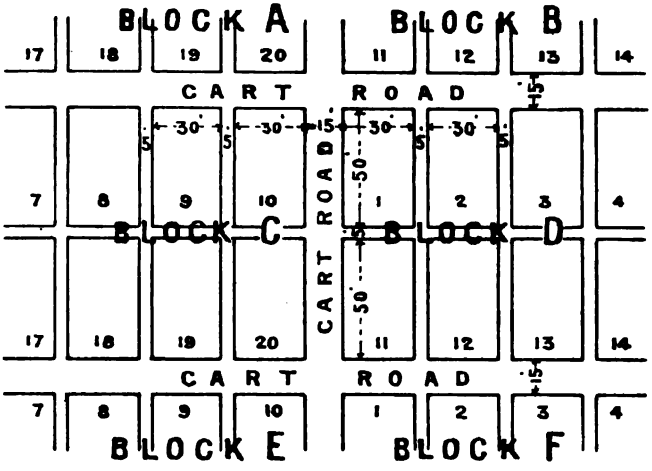
231. Works Arrangements.—A large scale plan of the whole of the works and working area should be made, so that everything may be shown on it and may be arranged with proper reference to its importance in respect to the whole scheme.

The most difficult and anxious part of the whole work is the closure of the dam, and everything should give place to this. Works roads should be set out with reference to it; all materials should be stacked, and earth and murum required for its rapid construction should be reserved for it, close at hand.

The area to be excavated for the construction of the dam

should be set out methodically, so as to get the maximum amount of material from it at the cheapest rate, while, at the same time, ensuring a check on the measurements. For this purpose some such sort of arrangement as is sketched in Fig. 53 should be made. The whole area should be first divided into blocks separated by roads for carts, and each block should be then subdivided into pits

FIG_53.



SCALE 80FT.=1 INCH

PLAN OF WORKING AREA

to be excavated. At first these pits should be excavated uniformly from 3 feet to 5 feet in depth, and should be then deepened gradually in steps, 1 or 2 feet deep, from the cart road, so that the carts may enter them by moving down a ramp left at one side. An index plan with reference numbers should be kept for each block, which should itself have a reference letter, in order that the measurements may be easily traced at any time.

These pits are likely to be fouled by the workers, and a strict watch should be kept on them to prevent this from occurring. The most effectual preventive is, however, to arrange for proper latrine areas, at distances conveniently near to the working area, and to conserve them properly.

For all special works men should be specially selected and trained. The general arrangements should ensure that each evening every responsible man is fully acquainted with the work which he has to get done the following day; alterations of this programme should be avoided as much as possible, as they cause confusion and delay, and thus occasion expense and retard progress.

III. Plant Required for the Works.

232. General Remarks.—In Bombay the earlier works were constructed with very little plant, reliance being placed upon labourers, pack animals, and carts for the conveyance of the earthwork on to the dam. The objections to this are that progress is not so rapid as might be, and that there is a greater liability to strikes and stoppage of work. The animals and carts no doubt help in the consolidation of the earthwork, but this can be done as effectually by rolling. General arrangements for "stores and tools" and "plant" are described under those headings in Appendix 22.

233. Tram Plant.—For the rapid conveyance of earthwork it is advisable to have tramway plant consisting of light trucks, and rails with sidings, points, cross-overs, &c., and there should be sufficient of this stocked, so that some may be always available from store to replace damaged articles or to meet an emergency requiring greater progress. The trucks should be light ones, chiefly of the side-tip pattern, with a few of the end-tip variety; the fittings should be as simple as possible, so that they may be renewed

on the works when necessary and may not be easily injured. The trucks can be pushed by labourers or hauled by animals: it may in certain cases, such as the construction of the gorge embankment, be useful to employ traction engines and wire cables to haul them.

In constructing the dam with tram plant, the rails should be laid on it parallel to its axis, and should first be placed near one edge; the lines should then be shifted gradually over the top of the dam until they approach the other edge. As they are moved on, the earth deposited from the trucks should be levelled, mixed, rolled, and finally wetted, each operation taking place on strips parallel to the axis of the dam.

Rules for the management of tram plant are given in Appendix 22, part 5, paragraph 59.

234. Rollers.—A light roller, weighing, say, a quarter of a ton per foot run, should be first used to consolidate and form on the deposited earth a surface upon which the ordinary roller, weighing three-quarters of a ton per foot run, can work. Finally, in the case of all high and important dams, it is desirable to complete the consolidation by means of a 10-ton steam roller. Grooved rollers, split rollers, and light stone rollers for the top of the dam have been noticed in Chapter II., paragraph 119. Ordinary rollers should be of cast iron, not stone, and should be provided with scrapers to prevent them from lifting the earth.

235. Pumps.—Water should be laid on all over the works both for constructional purposes and for the supply of the labourers. As the lifts will be high for large dams, it is advisable to have for them steam pumps, piping, and 4-feet cube iron tanks connected in series as reservoirs for these purposes. Hand pumps are useful for lower lifts and for unwatering foundations; for the latter there should be a reserve of pumping power to deal with any unusual amount

of water which may be met with, as in such cases everything depends upon the rate at which it can be got rid of.

236. Stone-Metal Crushers and Concrete Mixers.—These will be required only when there is a large amount of concrete to be made for the waste-weir and outlet; as a general rule, hand labour will suffice for the preparation of material for these works.

237. Tools and Instruments (*vide* Appendix 22, part 4).—It is true economy to have good tools, especially on a large work, where it is certain they will be worn out rather than will rust away. There should always be a large balance of good tools in store to meet sudden demands, and to enable useless tools to be replaced without delay.

Similarly, there should be a reserve of surveying instruments, which are peculiarly liable to injury on a large work.

IV. Maintenance of Works.

238. The Dam.—During the construction of the dam, careful observations by level and theodolite should be made, as explained in Chapter II., paragraph 125, at all high parts of the dam, and at such others where from any cause there may be any tendency to failure. These observations should be continued for at least a year after all movement has apparently ceased. During their continuance and after their cessation an annual record should be maintained of the levels of the top of the dam taken on the chainage stones fixed on it.

The next most important matter to which attention should be paid is the drainage of the dam. Water should not be allowed to concentrate when flowing down the slopes nor to lodge anywhere near its base, and all drains there may be should be kept running free from all obstructions. Wherever possible, a continuous record of the

discharges of such drains should be maintained as a permanent record. The best test of the sufficiency of the drainage is that the ground downstream of the dam is dry.

The slopes of the dam should be kept always clear of long grass and shrubs, the latter being carefully rooted out. Some weeks before the monsoon commences, the whole of the slopes should have the vegetation on them burned; this will make the young grass spring up the better afterwards, and will also expose any rat-burrows, &c., and give time for their being cut out and refilled. During the monsoon a permanent gang of labourers should be employed in preventing the guttering of the slopes and in filling up cracks and rain scores. This gang should be maintained at full strength for the first two monsoons, and then gradually can be reduced as experience dictates.

The pitching should be examined as the water level falls, and all loose stones, settlements, and other defects made good in a continuous system of repair.

239. The Waste-Weir.—If there is a temporary crest, this should be removed as soon as the reservoir falls below permanent crest level, and all woodwork in connection with it should be dried, tarred, and stored. It is generally not advisable to attempt further storage from occasional storms later on in the year.

As soon as the sluices are laid dry, they should be oiled and painted, and, some time before the monsoon commences, the lifting gear should be tested, so as to see all is in perfect working order.

The masonry should have all repairs effected as early as possible after the monsoon, so that the mortar may have the advantage of setting properly in the cold weather. All plants growing on it should be continuously rooted out.

240. The Outlet.—The culvert should be examined twice a year, some little time before the monsoon and just

after its close, and all structural repairs should be then made good. The valves, their seats and their rods, should be examined a month before the monsoon, and all parts in contact greased and others painted. The capstan heads and screws should be kept constantly oiled and protected from the weather; in the case of the screws this may be done by wrapping them round with oiled coir string. The ironwork of the approach bridge and headwall should be kept continuously painted, and any settlement which may take place at the approach bank should be at once made up.

241. The Reservoir.—The plantations should be carefully attended to, and preparations for planting more trees made throughout the fair weather by the small number of guards maintained for watching the plantations.

If silt clearance by ploughing is attempted (Chapter I., paragraph 36), the ploughing should proceed continuously as the reservoir level falls and the ground dries sufficiently to permit of it. Field owners should be encouraged to embank their fields in order to catch silt; where there is not hard ground for the formation of small waste-weir escapes, pipes with proper inlets might be fixed at the flanks to lead the impounded water safely away.

The silt experiment lines (Chapter I., paragraph 39) should be examined, and should be levelled over just before the replenishment of the reservoir is likely to commence.

V. The Report, Plans, Estimates, and Specifications.

242. The Report.—(*a*) *General.*—The Report forwarding the plans and estimates for sanction should be as concise as possible, and repetitions should be avoided. The writer should, however, place himself in the position of the persons

who will have to study the Report without the advantage of detailed local knowledge, and should make everything as clear to them as it is to himself from possessing that local knowledge. All statistical information should be given in a table, as in Appendix 26, where it can be found in a moment, whereas, if it is buried in a mass of verbiage, it is not so easily traceable. It is not necessary that the Report should discuss matters of common professional knowledge, as its main objects are to explain the reasons for selecting the site dealt with and rejecting others, and to describe the particular features of the scheme. The reasons for the rejection of these other sites should be carefully given, so as to show that the whole neighbourhood has been thoroughly investigated, and thus to avoid a reference on this point by those who have to examine the scheme.

(*b*) *Preparation of the Project.*—The extent of survey done is best explained by recording the result of the field work on the plans. The names of all those who have been connected with the drawing up of the scheme should be reported, so that they may be given all the credit which is their due, and so that proper weight may be attached to their opinions.

(*c*) *The Site and the Works.*—All peculiarities of the site; the nature of the foundations; the reasons for locating the different works at the sites settled for them, and for choosing the types of works adopted in the project; and the extent, position, and cost of the materials required for construction should be described.

(*d*) *Statistics of Rainfall and Yield.*—In respect to rainfall statistics, it is best to tabulate the daily fall during the monsoon months of as many years as possible, rather than the total monthly or annual falls. From such a table may be prepared one showing falls each over 1 inch, which alone are likely to produce a fair amount of run-off. The

statistics on which the yield of the catchment has been calculated should be given in detail, and it should be explained if rainfall records or river-gauge statements of discharge have been depended upon—the latter, it is scarcely necessary to say, being the better of the two, as they show the actual results of all the factors producing run-off.

(*e*) *Waste-Weir Floods.*—The disposal of the waste-weir floods should be described, and it should be made perfectly clear that their course will be harmless to neighbouring lands and property, or, if they are likely to cause damage, that full provision for compensation has been made in the estimates.

(*f*) *Revenue Matters.*—The principal crops grown in the neighbourhood; the probable effect on cultivation of the introduction of irrigation; the amount of manure and fuel available; the character of the cultivators; the peculiarities of the seasons and climate; the prospects of trade owing to the nearness of good markets; and the facilities afforded by good communications by road and rail should be stated, and the opinions of the Revenue officers on all these matters should be quoted.

(*g*) *Financial Return.*—When calculating the financial return, the results of an average year should be taken into account, that is to say, the storage, "duty," and area irrigated which are considered should be those of an ordinary year. The protective value of the work during a drought year should be also estimated.

(*h*) *Arrangement of Report.*—The Report should be divided into sections and paragraphs, and these should be carefully indexed for ready reference. At the end should be a series of statements giving the principal dimensions, costs, &c., of the project (Appendix 26); the reservoir contents at each contour (Appendix 17); rainfall and river gauge statistics; and waste-weir flood calculations (Appendices 12, 13 and 14).

243. The Plans.—Care should be taken to send up a complete set of plans, so that the scope of the project may be easily ascertained from them. It is not necessary to waste time in making any but type drawings for minor works, as these may have to be altered during construction, and their detailed design can be safely entrusted to the discretion of the officers who will have to carry them out. The scales of the drawings should be chosen so that they may be as small as is consistent with showing the proper amount of detail in them; large, unwieldly drawings are difficult to deal with, and are usually unnecessary, as intricate parts can be easily shown by enlarged detailed drawings. The index plan should show all places mentioned in the Report; all roads, railways, irrigation works, and natural features; and the proposed canal system and the land irrigable by it.

The following is a list of drawings, which comprises all that will be generally required :—

1. Index Plan of Project.—Foolscap size (for insertion in the Report).

2. General Plan of Catchment.—Scale, 1 mile to 1 inch,

3. Contoured Plan of Reservoir.—Scale, 660 feet to 1 inch, (to show all works and the waste-weir outfall).

4. Land Plan (to show all land to be acquired).—Scale, 660 feet to 1 inch, (this should correspond with the village maps).

5. Dam.—Plan, longitudinal and cross sections; details of foundations and closure arrangements.

6. Waste-Weir.—Plan, longitudinal and cross sections; details of automatic gates, sluices, and temporary weir crest.

7. Outlet.—Plan, longitudinal and cross sections; details of tail- and fore- bays, approach bridge, valves, lifting rods, and capstans.

On every drawing practicable all the foundations and the trial pits by which they were determined should be shown; the direction of the flow of water should be indicated by arrows; and all reduced levels, water levels, and dimensions should be carefully given, so that they may be seen at once on inspection.

Each plan should bear the name of the project and its own distinctive name; the year of its preparation; the estimated cost of the work; the names of the surveyor, designer, and draughtsman; and references to benchmarks and to the pages of the book in which the survey is recorded. It will be convenient if the plans (tracings or blue prints), are folded foolscap size and bound in a series of pamphlets (or placed in small portfolios), for easy reference, each pamphlet, or portfolio, having outside an index to the drawings it contains.

244. The Estimates.—The estimates should be prefaced by a Recapitulation showing the total cost of each work arranged under main heads (*vide* Appendix 15).

The estimates of the works should then follow in the order of the recapitulation, and each should consist of a general description, the detailed measurements and an abstract of cost. The general description should be confined to explaining the drawings and to giving any tabular information there may be respecting them: the reasons for selecting the type of work chosen and other general particulars should appear in the Report itself. The measurements under the different subheads should be arranged throughout in the same order, so that the total quantities and costs of any particular part of the work may be ascertained at any time. The abstract should be made out so as to exhibit clearly the total cost of each subwork, and the "contingencies" allowed for each should be added in each case to enable this to be done (*vide* Appendix 16).

245. The Specification.—It is desirable to draw up for each district a complete specification which can be printed and attached to each project, thus saving trouble and securing as much uniformity as is desirable. The specification should clearly distinguish between what is definitely settled and what is left to the discretion of the principal responsible officer, and as much latitude as possible should be given to him to meet unforeseen requirements. A specimen specification for earthwork and pitching is given in Appendix 18.

ENDIX 1.[1]

S WITH EARTH

HAPTER I., PARAGRA

r.	Area of Co millions of s	Remarks.
	At full-supply level.	
	17	29
... ...	22·177	for Lower Pánjhra River Works.
... ...	6·575	ater-supply.
... ...	18·348	
... ...	2·057	k restored.
... ...	6·620	
... ...	36·000	
... ...	21·000	
... ...	15·000	is the originally designed length of the waste- nd 400 feet is its actual length.
... ...	6·750	ater-supply.
... ...	2·000	)o.
... ...	198·232	
... ...	123·296	
... ...	8·530	ater-supply.
sonry ...	174·840	
... ...	29·490	for Yerla Canals.
masonry	15·220	Gondoli Canal
nd flood	16·570	
... ...	3·365	ater-supply.
channel	3·373	pply for Sátára; small scheme abandoned for .
n ...	7·360	

e run-off bears to the whole upply contour, the distance

APPENDIX 2.

TABLE OF TOTAL MONSOON RAINFALL AND ESTIMATED RUN-OFF AND YIELD PER SQUARE MILE FROM CATCHMENT AREAS.

(*Vide* Chapter I., paragraph 16.)

1	2	3	4	5	6	7	8	9	10
	Good Catchment.			Average Catchment.			Bad Catchment.		
Total Monsoon Rainfall in inches.	Per-centage of Run-off to Rainfall.	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. cft.	Per-centage of Run-off to Rainfall.	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. cft.	Per-centage of Run-off to Rainfall.	Depth of Run-off due to Rainfall in inches.	Yield of Run-off from Catchment per square mile in mill. cft.
1	0·1	0·001	0·002	0·1	0·001	0·001	0·05	0·0005	0·001
2	0·2	0·004	0·009	0·15	0·003	0·006	0·1	0·002	0·004
3	0·4	0·012	0·028	0·3	0·009	0·021	0·2	0·006	0·014
4	0·7	0·028	0·065	0·5	0·021	0·048	0·3	0·014	0·032
5	1·0	0·050	0·116	0·7	0·037	0·087	0·5	0·025	0·058
6	1·5	0·090	0·209	1·1	0·067	0·156	0·7	0·045	0·104
7	2·1	0·147	0·341	1·5	0·110	0·255	1·0	0·073	0·170
8	2·8	0·224	0·520	2·1	0·168	0·390	1·4	0·112	0·260
9	3·5	0·315	0·732	2·6	0·236	0·549	1·7	0·157	0·366
10	4·3	0·430	0·999	3·2	0·322	0·749	2·1	0·215	0·499
11	5·2	0·572	1·329	3·9	0·429	0·996	2·6	0·286	0·664
12	6·2	0·744	1·728	4·6	0·558	1·296	3·1	0·372	0·864
13	7·2	0·936	2·174	5·4	0·702	1·630	3·6	0·468	1·087
14	8·3	1·162	2·699	6·2	0·871	2·024	4·1	0·581	1·349
15	9·4	1·410	3·276	7·0	1·057	2·457	4·7	0·705	1·638
16	10·5	1·680	3·903	7·8	1·260	2·927	5·2	0·840	1·951
17	11·6	1·972	4·581	8·7	1·479	3·435	5·8	0·986	2·290
18	12·8	2·304	5·353	9·6	1·728	4·014	6·4	1·152	2·676
19	13·9	2·641	6·135	10·4	1·980	4·601	6·9	1·320	3·067
20	15·0	3·000	6·970	11·25	2·250	5·227	7·5	1·500	3·485
21	16·1	3·381	7·855	12·0	2·535	5·891	8·0	1·690	3·927
22	17·3	3·806	8·842	12·9	2·854	6·631	8·6	1·903	4·421
23	18·4	4·232	9·832	13·8	3·174	7·374	9·2	2·116	4·916
24	19·5	4·680	10·873	14·6	3·510	8·154	9·7	2·340	5·436
25	20·6	5·150	11·964	15·4	3·862	8·973	10·3	2·575	5·982
26	21·8	5·668	13·168	16·3	4·251	9·876	10·9	2·834	6·584
27	22·9	6·183	14·364	17·1	4·637	10·773	11·4	3·091	7·182
28	24·0	6·720	15·612	18·0	5·040	11·709	12·0	3·360	7·806
29	25·1	7·279	16·911	18·8	5·459	12·683	12·5	3·639	8·455
30	26·3	7·890	18·330	19·7	5·917	13·747	13·1	3·945	9·165
31	27·4	8·494	19·733	20·5	6·370	14·799	13·7	4·247	9·866
32	28·5	9·120	21·188	21·3	6·840	15·891	14·2	4·560	10·594
33	29·6	9·768	22·693	22·2	7·326	17·019	14·8	4·884	11·346
34	30·8	10·472	24·329	23·1	7·854	18·246	15·4	5·236	12·164
35	31·9	11·165	25·939	23·9	8·373	19·454	15·9	5·582	12·969
36	33·0	11·880	27·600	24·7	8·910	20·700	16·5	5·940	13·800
37	34·1	12·617	29·312	25·5	9·462	21·984	17·0	6·308	14·656
38	35·3	13·414	31·163	26·4	10·060	23·372	17·6	6·707	15·581
39	36·4	14·196	32·980	27·3	10·647	24·735	18·2	7·098	16·490
40	37·5	15·000	34·848	28·1	11·250	26·136	18·7	7·500	17·424
41	38·6	15·826	36·767	28·9	11·869	27·575	19·3	7·913	18·383
42	39·8	16·716	38·835	29·8	12·537	29·126	19·9	8·358	19·417
43	40·9	17·587	40·858	30·6	13·190	30·643	20·4	8·793	20·429
44	42·0	18·480	42·933	31·5	13·860	32·199	21·0	9·240	21·466
45	43·1	19·395	45·058	32·3	14·546	33·793	21·5	9·697	22·529
46	44·3	20·378	47·342	33·2	15·283	35·506	22·1	10·189	23·671
47	45·4	21·338	49·572	34·0	16·003	37·179	22·7	10·669	24·786
48	46·5	22·320	51·854	34·8	16·740	38·890	23·2	11·160	25·927
49	47·6	23·324	54·186	35·7	17·493	40·639	23·8	11·662	27·093
50	48·8	24·400	56·686	36·6	18·300	42·514	24·4	12·200	28·343
51	49·9	25·449	59·123	37·4	19·086	44·342	24·9	12·724	29·561
52	51·0	26·520	61·611	38·2	19·890	46·208	25·5	13·260	30·805
53	52·1	27·613	64·151	39·0	20·709	48·113	26·0	13·806	32·075
54	53·3	28·782	66·866	39·9	21·586	50·149	26·6	14·391	33·433
55	54·4	29·920	69·510	40·8	22·440	52·132	27·2	14·960	34·755
56	55·5	31·080	72·205	41·6	23·310	54·153	27·7	15·540	36·102
57	56·6	32·262	74·951	42·4	24·196	56·213	28·3	16·131	37·475
58	57·8	33·524	77·883	43·3	25·143	58·412	28·9	16·762	38·941
59	58·9	34·751	80·734	44·1	26·063	60·550	29·4	17·375	40·367
60	60·0	36·000	83·635	45·0	27·000	62·726	30·0	18·000	41·817

NOTE.—Plate 1 shows these results diagrammatically

APPENDIX 3.

CALCULATION OF THE AMOUNT OF STORAGE REQUIRED FOR THE IRRIGATION OF A CERTAIN AREA AND TO SUPPLEMENT THE DISCHARGE OF A NATURAL STREAM.

(*Vide* Chapter I., paragraph 26.)

1. The following calculation shows how the amount of storage which is required to supplement the discharge of a natural stream may be determined. The stream considered is one which has a deficient supply in the fair season, and a superabundant one during the monsoon. By storing only sufficient of the excess discharge during the latter period to tide over the deficiency of the former one, the catchment can be made to serve the area contemplated at the minimum expense, as the natural daily discharge of the stream will thus be fully utilised.

2. The first thing to be done is to calculate the daily supply required by the canal and then to gauge the daily discharge of the stream, as the irregularity of the flow of the latter will not permit of monthly results being taken directly into account. When the discharge of the stream is in excess of the requirements of the canal, the balance is available for being stored. When, however, the former is less than the latter, the difference will have to be supplied by the storage already effected.

3. These daily observations should be recorded in the following form :—

Register of Daily Gaugings.

1	2	3	4	5
Date.	Discharge required for Canal. Average for day. Cft. per sec.	Discharge of Feed Channel at Weir Site. Average for day. Cft. per sec.	Discharge required from Storage. Average for day. Cft. per sec.	Discharge that can be Stored. Average for day. Cft. per sec.

At the end of the month the columns should be totalled and multiplied by 86,400, the number of seconds in one day, and the results thus obtained tabulated in the form given below:—

ESTIMATE OF STORAGE REQUIRED.

1	2	3	4	5	6
Year and Month.	Quantity required for Canal. Mill. cft.	Discharge of Feed Channel at Weir Site. Mill. cft.	Quantity required from Storage. Mill. cft.	Quantity that can be Stored. Mill. cft.	Contents in Tank, with F.S. Storage = 150 mill. cft. (at end of each month). Mill. cft.
			Initial	storage ...	130·373
January ...	26·784	5·575	21·209	,,	109·164
February...	25·056	4·150	20·906	,,	88·258
March ...	26·784	1·584	25·200	,,	63·058
April... ...	25·920	10·146	22·397	6·623	47·284
May	26·784	0·453	26·331	,,	20·953
June	39·744	25·871	32·587	18·714	7·080
July	53·568	19·982	40·576	6·990	**26·506**
August ...	53·568	35·717	39·359	21·508	**44·357**
September..	51·840	10·262	41·578	,,	**85·935**
October ...	38·880	68·947	25·799	55·866	30·067
November..	25·920	4·881	21·039	,,	9·028
December..	26·784	16·045	20·876	10·137	1·711
Totals ...	421·632	203·613	337·857	119·838	

NOTES.

1. Col. 2 + col. 5 = col. 3 + 4.

2. The increments in col. 6 are equal to the excesses of col. 5 over col. 4, and the decreases, to those of col. 4 over col. 5.

3. In col. 6 the entries in light type indicate the contents at the ends of the months of a tank with an assumed F.S. Storage of 150 mill. cft., no allowance for evaporation being made. The entries in heavy type indicate the additional storage required in the tank to enable it to tide over the periods concerned.

	Mill. cft.
4. The total of col. 3	203·613
Plus the initial storage	130·373
Minus the final storage (in this case a minus quantity)	1·711
Plus the maximum deficiency	85·935
Equal the total of col. 2	421·632

5. To ascertain the amount of storage required:—to the assumed storage add the maximum deficiency, or, deduct the maximum excess, and then add the proper allowance for evaporation and absorption.

APPENDIX 4.

TANK EXPENDITURE ESTIMATE.

(*Vide* Chapter I., paragraph 27.)

1. For the proper regulation of the draw-off from a Tank it is necessary to frame an estimate of expenditure, month by month. The following data may be assumed:—

2. **Duty of Water.**—This may be taken thus:—

Nov. 1st—Feb. 28th	...	60 acres per cft. per. sec.
March 1st—June 30th	...	40 „ „ „
July 1st—Oct. 31st	...	80 „ „ „

Note 1.—The actual seasons begin and end fifteen days earlier than these dates, but this may be neglected for the estimate.

Note 2.—The duties are purposely taken low so as to agree with what occurs in practice; by good management it should be easy to increase them.

Note 3.—The duties assumed are average ones for all classes of crops that may be irrigated at the same time.

3. **Acreage under Irrigation.**—It will be necessary to assume the acreage under irrigation from time to time, and the utility of the estimate will depend upon the correctness of the assumptions made. The acreage assumed should be as large as previous experience indicates, and subject to the restriction noted in paragraph 7 below.

4. **Amount of Draw-off.**—This should be calculated on the acreage assumed and on the duty given in paragraph 2 above.

5. **Allowance for Evaporation and Absorption, &c.**—This may be assumed thus:—

Nov. 1st—Feb. 28th	...	2 in. depth per month over the top area of the Tank.
March 1st—June 30th	...	6 in. „ „
July 1st—Oct. 31st	...	4 in. „ „

These amounts should be deducted from the estimated Tank level at the end of each month so as to give the estimated level at the beginning of the next month. They correspond to a loss of storage during the year which is equal to a draw-off of 4 feet in vertical depth from the mean area of the Tank.

6. **Available Supply.**—The amounts available at the different Tank levels should be taken from the table of Tank contour capacities. The necessary deductions for the silting-up of the Tank should be made.

7. **Extent of Estimate.**—The estimate should extend from the time when there is no more chance of replenishment, say November 1st, until there is a fair certainty of monsoon replenishment during the next monsoon, as ascertainable from the records. It should start with the actual level of the Tank. The estimated future levels, when once sanctioned, should not be exceeded without further sanction, which should be obtained after giving full explanation of the causes necessitating it. The estimate should close with a small balance in the Tank to allow for contingencies, and, for the same reason, credit should not be taken for hot-weather replenishments. It should be submitted on November 15th.

8. **Type Estimate.**—The following is given as part of a type estimate of a Tank supplying a Canal which is also partly fed by a River :—

Month.	At beginning of month.		River supply. Million cubic feet.	Total available supply. Million cubic feet.	Acres irrigated.	Duty allowed, acres per cubic foot per second.	Total consumption of water. Million cubic feet.	Consumption of water from Tank alone. Million cubic feet.	R.L. of Tank due to consumption.	Allowance for evaporation. Feet.	Estimated R.L. of Tank at end of month.	Remarks.
	Estimated R.L. of Tank.	Storage in Tank. Mill. cubic feet.										
1	2	3	4	5	6	7	8	9	10	11	12	13
Nov.	113·16	247·86	134·14	382·00	2,000	60	86·400	0·00	113·16	0·16	113·00	
Dec.	113·00	242·29	57·71	300·00	2,500	60	111·600	53·89	109·51	0·16	109·35	

APPENDIX 5.

ESTIMATE OF THE DURATION OF A WATER-SUPPLY STORAGE.

(*Vide* Chapter I., paragraph 27.)

THE following estimate was made of the duration of supply in a connected series of reservoirs designed for the water-supply of the town of Dharwar, Bombay Presidency. It was assumed in it that the storage of the upper reservoirs would be first utilised, being run-off for this purpose into the main Kelgeri Storage Reservoir, and that the latter would be the last to be drawn upon. For the sake of simplicity of calculation it was further assumed that the storage required to make good the loss by evaporation from the lower tanks was not replaced from the upper subsidiary tanks.

1	2	3	4	5	6	7	8	9	10	11
Month.	At commencement of Month.			Evaporation.		Draw-off.	Total depletion.	At end of Month.		
	Tank.							Tank.		
	R.L.	Surface area.	Contents.	Depth on top area.	Amount.			Contents.	R.L.	Surface area.
		Mill. sq. feet.	Mill. cft.	Feet.	Mill. cft.	Mill. cft.	Mill. cft.	Mill. cft.		Mill. sq. feet.
NAIKANKHEDI TANK.										
Nov.	F.S.L. 91·00	0·649	2·528	0·16	0·104	2·100	2·528	Nil, Tank run dry on Nov. 30th.		
	Balance in tank and allowed for contingencies				0·324					
KENCHANHATTI TANK.										
Nov.	F.S.L. 80·00	1·220	4·879	0·16	0·195	—	0·195	4·684	79·80	1·200
Dec. ...	79·80	1·200	4·684	0·16	0·192	2·170	2·362	2·322	79·50	0·775
	79·50	0·775	2·322	0·16	0·124					
Jan.	Balance in tank and allowed for contingencies				0·028	2·170	2·322	Nil, Tank run dry on Jan. 31st.		

1	2	3	4	5	6	7	8	9	10	11
Month.	At commencement of Month.			Evaporation.		Draw-off.	Total depletion.	At end of Month.		
	Tank.			Depth on top area.	Amount.			Tank.		
	R.L.	Surface area.	Contents.					Contents.	R.L.	Surface area.
		Mill. sq. feet.	Mill. cft.	Feet.	Mill. cft.	Mill. cft.	Mill. cft.	Mill. cft.		Mill. sq. feet.
Kelgeri Urmundinkeri.										
Nov.	F.S.L. 78·76	2·932	12·106	0·16	0·469	—	0·469	11·637	78·60	2·870
Dec. ...	78·60	2·870	11·637	0·16	0·459	—	0·459	11·178	78·44	2·790
Jan. ...	78·44	2·790	11·178	0·16	0·446	—	0·446	10·732	78·28	2·730
Feb. ...	78·28	2·730	10·732	0·33	0·910	1·960	2·870	7·862	77·15	2·320
March ...	77·15	2·320	7·862	0·67	1·554	2·170	3·724	4·138	75·25	1·680
	75·25	1·680	4·138	0·67	1·120					
April	Balance in bed of tank and allowed for contingencies ...				0·918	2·100	4·138	Nil, Tank not drawn on after April 30th.		
Kelgeri Storage Reservoir										
Nov.	F.S.L. 60·00	6·599	59·299	0·16	1·056	—	1·056	58·243	59·84	6·530
Dec. ...	59·84	6·530	58·243	0·16	1·045	—	1·045	57·198	59·68	6·470
Jan. ...	59·68	6·470	57·198	0·16	1·035	—	1·035	56·163	59·52	6·400
Feb. ...	59·52	6·400	56·163	0·33	2·133	—	2·133	54·030	59·19	6·280
March ...	59·19	6·280	54·030	0·67	4·120	—	4·120	49·910	58·52	6·000
April ...	58·52	6·000	49·910	0·67	4·000	—	4·000	45·910	57·85	5·770
May ...	57·85	5·770	45·910	0·51	2·942	2·170	5·112	40·798	56·90	5·420
June ...	56·90	5·420	40·798	0·31	1·807	2·100	3·907	36·891	56·16	5·154
July ...	56·16	5·154	36·891	0·16	0·825	2·170	2·995	33·896	55·50	4·935
Following June	55·50	4·935	33·896	3·48	11·091	23·360	34·471	Nil, Tank dry June 30th.		
At pumping sill ...		1·437		During 11 months.						
Mean area ...		3·187								

Notes.

1. These calculations show that, starting with full tanks on November 1st, the supply would last up to June 30th of the next year but one, and that they would thus tide over a year during which there was no replenishment.

2. The allowances for evaporation and absorption, on the *top areas*, are:

	Nov.	Dec.	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.
For	0·16	0·16	0·16	0·33	0·67	0·67	0·51	0·31	0·16	0·16	0·16	0·17,

i.e., for the whole year 3·65 feet. A separate allowance was not estimated for absorption, but it is believed it will be very small, as the beds of all of the tanks are formed of puddled rice fields. On the other hand, full credit has not been given for the rain falling on the tanks themselves, which would compensate for a good deal of loss on this account.

3. The daily supply to be given to the town is 430,000 gallons, or 68,900 cubic feet. In the calculations it has been taken equal to 70,000 cubic feet.

APPENDIX 6.

STATISTICS OF CERTAIN IRRIGATION WORKS IN INDIA.

(*Vide* Chapter I., paragraph 29—31.)

TABLE I.

DUTY OF WATER OBTAINED ON CERTAIN IRRIGATION WORKS IN INDIA DURING 1885—86, 1890—91, AND 1895—96.

(*Vide* Irrigation Revenue Report Statistical Table I.—E.)

Province.	Name of Work.	Acres irrigated per cubic foot of discharge utilised.					
		1885—86.		1890—91.		1895—96.	
		Kharif.	Rabi.	Kharif.	Rabi.	Kharif.	Rabi.
1	2	3	4	5	6	7	8
	1. TANKS.						
Bombay ...	1. Mhasvád Tank	148·00	43·00	88·54	52·41	41·64	41·21
	2. Ekruk Tank	50·83	28·07	57·77	29·38	60·76	31·05
United Provinces	3. Jhansi Lakes	—	—	—	21·00	7·00	72·00
	4. Hamirpur Lakes	—	29·00	—	31·00	7·00	37·00
Rajputana	5. Bir Tank	—	—	—	—	10·00	36·00
	6. Ladpura Tank	—	—	—	—	49·00	51·00
	7. Balad old and new Tanks	—	—	—	—	18·00	22·00
Baluchistan	8. Khushdil Khan Reservoir	—	—	—	—	—	138·66
	2. CANALS.						
Bombay ...	9. Nira Canal	115·00	60·00	76 24	56·29	54·09	54·22
	10. Hathmati Canal	16·20	47·44	40·84	67·56	36·36	54·00
	11. Krishná Canal	61·00	22·00	38·00	37·00	42·00	25·00
Sind ...	12. Mithrau Canal	51·55		34·37		37·58	28·37
	13. Sukkur Canal	29·85		33·98		37·17	240·17
United Provinces	14. Upper Ganges Canal ...	86·00	171·00	81·00	142·00	66·00	123·00
	15. Agra Canal	97·00	131·00	95·00	115·00	68·00	105·00
	16. Betwa Canal	—	65·00	28·00	77·00	28·00	96·00
Punjab ...	17. Swat River Canal ...	—	158·00	57·00	126·00	77·00	116·00
	18. Bari Doab Canal	64·00	157·00	64·00	140·00	68·00	135·00
	19. Sirhind Canal	102·00	193·00	66·00	169·00	44·00	156·00
Bengal ...	20. Orissa Canals	80·10	—	90·99	11·32	67·00	—
	21. Sone Canals	94·30	99·90	88·48	47·41	74·32	72·87
Baluchistan...	22. Shebo Canal	—	—	35·00	94·00	20·80	99·70
Madras ...	23. Godavari Delta System...	90·45	7·04	94·93[1]	8·88[2]	94·28[1]	27·82[2]
	24. Cauvery ,,	71·34	58·37	72·21[1]	1,204·61[2]	61·94[1]	90·73[2]
	25. Kistna ,,	78·24	—	124·44[1]	—	125·05[1]	—

[1] First Crop. [2] Second Crop.

For Sind and Madras the figures given are the areas irrigated per cubic foot of discharge at head.

TABLE II.

DUTY OF WATER OBTAINED ON CERTAIN IRRIGATION WORKS IN BOMBAY (DECCÁN), GIVING AVERAGE RESULTS.

(*Vide* Irrigation Revenue Report, Statistical Table I.—E.)

Name of System.	Utilised Discharge.													
	Kharif duty.							Rabi duty.						
	1894—95	1895—96	1897—98	1898—99	1900—01	Total.	Average.	1894—95	1895—96	1897—98	1898—99	1900—01	Total.	Average.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1. Mhasvád Tank	34	42	13	73	109	271	54	50	41	101	42	51	285	57
2. Ekruk Tank	61	61	86	83	210	501	100	38	31	44	44	—	157	39
3. Bhátodi Tank	122	19	118	14	82	355	71	135	123	154	101	62	635	127
4. Nirá Canal	48	54	100	85	153	440	88	57	54	124	91	118	444	89
5. Krishná Canal	49	42	52	38	137	318	64	22	25	26	26	53	152	30
6. Lower Pánjhrá Canals ...	103	94	77	375	—	649	162	—	132	—	—	142	274	137
Totals ...	417	312	446	668	691	2,534	539	302	406	449	364	426	1,947	479
Average ...	70	52	74	111	138	445	89	60	68	90	75	85	378	76

The years 1896—97 and 1899—1900 are excluded, being famine years.

TABLE III.

PROPORTIONS OF AREAS IRRIGATED TO CULTURABLE AREAS UNDER COMMAND ON IRRIGATION WORKS IN BOMBAY (DECCÁN).

(*Vide* Irrigation Revenue Reports, Statistical Table IV.—E.)

Year.	Culturable area in acres.	Irrigated area in acres.	Percentage of col. 3 to col. 2.	Remarks.
1	2	3	4	5
1888—89	512,301	79,195		The entries in col. 2 for the years 1888—89 to 1893—94 are believed to show the total culturable area under command of the completed projects (*vide* Irrigation Revenue Report, 1894—95, para. 6 of G. R., No. 9, W.I., 123 of Jan. 23rd, 1896). For the remaining years the entries in col. 2 show the culturable area under command of the projects as actually constructed.
1889—90	533,313	86,599		
1890—91	535,762	75,901		
Total ...	1,581,376	241,695	15·3	
1891—92	559,911	97,074		
1892—93	571,903	66,486		
1893—94	583,897	81,000		
Total ...	1,715,711	244,560	14·3	
Grand Total, 1888—1894	3,297,087	486,255	14·7	
1894—95	341,015	85,394		
1895—96	315,040	76,129		
1896—97	279,741	119,210		
Total ...	935,796	280,733	30·0	
1897—98	316,425	127,722		
1898—99	316,425	105,055		
1899—1900	316,425	105,830		
Total ...	949,275	338,607	35·7	
Grand Total, 1894—1900	1,885,071	619,340	32·9	
1900—01	316,425	126,178	39·9	
Grand Total, 1894—1901	2,201,496	745,518	33·9	

TABLE IV.

AREAS IRRIGATED UNDER DIFFERENT CROPS IN BOMBAY (DECCÁN).

(*Vide* Irrigation Revenue Report, Statistical Table III.—E.)

Description of Crop.		Acres Irrigated.						
		1894—95.	1895—96.	1897—98.	1898—99.	1900—01.	Total.	Average.
1		2	3	4	5	6	7	8
1. Perennial	Area ...	*17,633	14,533	10,198	11,094	10,766	64,224	12,845
	Percentage	22	20	8	11	9	13	13
2. Rabi	Area ...	*14,823	*12,035	21,023	25,120	21,521	94,522	18,904
	Percentage	18	17	17	25	19	19	19
3. Monsoon dry	Area ...	34,788	33,459	*75,198	48,812	*71,167	263,424	52,685
	Percentage	43	46	62	49	61	54	54
4. Eight months	Area ...	12,393	10,596	11,702	12,762	12,196	59,649	11,930
	Percentage	15	15	10	13	10	12	12
5. Hot weather	Area ...	1,117	1,101	*3,332	1,538	1,783	8,871	1,774
	Percentage	2	2	3	2	1	2	2
6. Total	Area ...	80,754	71,724	121,453	99,326	117,433	490,690	98,138
	Percentage	100	100	100	100	100	100	100

TABLE IVA.

AREAS AND PERCENTAGES OMITTING EXCEPTIONAL AREAS, MARKED THUS * IN TABLE IV.

Description of Crop.	Acres Irrigated.		
	Total.	Average.	Percentage.
1	2	3.	4.
1. Perennial	46,591	11,658	15
2. Rabi	67,664	22,555	23
3. Monsoon dry	117,059	39,020	40
4. Eight months	59,649	11,930	20
5. Hot weather	5,539	1,385	2
6. Grand total	296,502	—	100

The years 1896—97 and 1899—1900 are excluded, being famine years.

APPENDIX 7.

(*Vide* Chapter I., paragraphs 28 and 32.)

ESTIMATES OF REVENUE RESULTS FROM TANK STORAGE.

1. Estimate of the Irrigating Capacity of Tank Storage.

Let A = the area in acres of each crop in quadrennial rotation;
Q = the quantity in cubic feet required for the irrigation of 4 A;
The duty for perennial crops (sugar, plantains, &c.)=100 acres for 365 days;
The duty for rice = 40 acres for 122 days;
The duty for monsoon dry crops = 160 acres for 122 days;
The duty for rabi = 120 acres for 121 days.

	Discharge, cft. per sec.	Secs.	Days.	Cubic feet.
Then Q =	$\frac{A}{100}$ ×	86,400 ×	365 =	315,360A
+	$\frac{A}{40}$ ×	86,400 ×	122 =	263,520A
+	$\frac{A}{160}$ ×	86,400 ×	122 =	65,880A
+	$\frac{A}{120}$ ×	86,400 ×	121 =	87,120A
Q for 4 A . . .			=	731,880A, say 750,000A, cft.

adding one-third for loss by evaporation and absorption = E,

Q + E for 4 A = 1,000,000 cubic feet.

Or, in other words, if A = 1 acre, with these duties, the tank will irrigate 4 acres per million cubic feet of storage.

If the duties are taken three-quarters of the above, the tank will irrigate 3 acres per million cubic feet of storage.

If the duties are taken one-half of the above, the tank will irrigate 2 acres per million cubic feet of storage.

2. Estimate of the Return from Tank Storage.

The following table shows the return which may be expected from the storage of one million cubic feet, assuming :—

That the quoted rates are assessed for the irrigation of the crops :

(1) when the work is first opened, and

(2) after it has been in operation for some years ;

that working expenses are at the rate of Rs.2 per acre ;

and that, respectively, full, three-quarter and half duties are obtained.

Estimate of Revenue from a Storage of One Million Cubic Feet.

1	2	3	4	5	6	7	8	9
Crop.	At Original Rates.				At Final Rates.			
	Rate per acre.	Revenue.			Rate per acre.	Revenue.		
		Whole duty.	¾ duty.	½ duty.		Whole duty.	¾ duty.	½ duty.
	Rs.	Rs.	Rs.	Rs.	Rs.	Rs.	Rs.	Rs.
1. Perennial	10	10·00	7·50	5·00	16	16·00	12·00	8·00
2. Rice	4	4·00	3·00	2·00	6	6·00	4·50	3·00
3. Monsoon dry ...	1	1·00	0·75	0·50	1	1·00	0·75	0·50
4. Rabi	3	3·00	2·25	1·50	4	4·00	3·00	2·00
5. Gross revenue ...	—	18·00	13·50	9·00	—	27·00	20·25	13·50
6. Working expenses ...	2	8·00	6·00	4·00	2	8·00	6·00	4·00
7. Net revenue	—	10·00	7·50	5·00	—	19·00	14·25	9·50

Applying the rates in force in any locality in this manner, it can be seen what will be the probable revenue from the storage ; and, estimating the approximate cost of the storage, the approximate return from the capital expenditure can be obtained.

3. Estimated Amount of Storage for Different Classes of Crops per Rupee of Assessment.

The following table shows the estimated amount of storage per rupee of assessment required for the different crops. This is deduced from :

(*a*) The estimated consumption of water utilised for bringing them to maturity, plus the estimated loss by evaporation which occurs from the storage until they are matured ; and

(*b*) the crop rates :—

ESTIMATED AMOUNT OF STORAGE PER RUPEE OF ASSESSMENT.

1	2	3	4	5	6	7	8	9	10
Crop.	Quantity of Storage Required.					At Original Rates.		At Final Rates.	
	Evaporation.			Storage utilised.	Total.	Assessment per acre.	Amount of Storage per rupee of Assessment.	Assessment.	Amount of Storage per rupee of Assessment.
	Loss until crop is matured.	Percentage loss.	Amount of loss.						
	Inches.		c. ft.	c. ft.	c. ft.	Rs.	c. ft.	Rs.	c. ft.
1. Perennial ...	48	46	115,000	325,000	440,000	10	44,000	16	27,500
2. Rice	16	16	40,000	270,000	310,000	4	77,500	6	51,667
3. Monsoon dry	16	16	40,000	65,000	105,000	1	105,000	1	105,000
4. Rabi	24	22	55,000	90,000	145,000	3	48,333	4	36,250
5. Totals ...	48 (during year)	100	250,000	750,000	1,000,000	—	—	—	—

The assessments are usually fixed with reference to the value of the crops, and not according to the amount of water which has to be stored for them. The above table shows, on the assumptions made, that under tanks the irrigation of valuable perennial crops should be encouraged, while that of monsoon crops should be discouraged so far as revenue and return are concerned. Unless, however, the land is heavily manured, a rotation of crops is necessary, and the return on the capital expenditure must be therefore estimated on the average results from the different crops under cultivation.

APPENDIX 8.

TABLE OF WASTE-WEIR RUNS-OFF.

(*Vide* Chapter III., paragraph 171.)

1	2	3	4	5	6
Increments of Catchment Area.	Run-off from each increment of Catchment Area in col. 1.	Discharge from each increment of Catchment Area in col. 1 due to Run-off.	Discharge from Total Catchment Area due to Run-off.	Average Run-off from Total Catchment Area.	Remarks.
Square miles.	Inches per hour.	Cubic feet per second.	Cubic feet per second.	Inches per hour.	
0—1	3·00	1,936	1,936	3·00	
1—2	2·65	1,710	3,646	2·83	
2—3	2·30	1,484	5,130	2·65	
3—4	2·00	1,291	6,421	2·49	
4—5	1·85	1,194	7,615	2·36	
5—6	1·72	1,110	8,725	2·25	
6—7	1·62	1,045	9,770	2·16	
7—9	1·52	981	10,751	2·08	
8—9	1·45	936	11,687	2·01	
9—10	1·40	903	12,590	1·95	
10—15	1·16	3,743	16,333	1·69	
15—20	1·00	3,227	19,560	1·51	
20—25	0·92	2,968	22,528	1·40	
25—50	0·80	12,907	35,435	1·10	
50—75	0·75	12,100	47,535	0·98	
75—100	0·70	11,293	58,828	0·91	
100—150	0·65	20,973	79,801	0·82	
150—200	0·60	19,360	99,161	0·77	

Col. 3 = Increment of area in Col. 1 × Col. 2 × 645·33 cubic feet.

Col. 4 = Sum of entries in Col. 3.

$$\text{Col. 5} = \frac{\text{Col. 4}}{\text{Last figure in Col. 1} \times 645{\cdot}33}.$$

NOTE.—Plate II. shows the entries in Cols. 2 and 5 diagrammatically.

APPENDIX 9.

TABLE OF VALUES OF BAZIN'S COEFFICIENTS FOR EARTHEN CHANNELS.[1]

(*Vide* Chapter III., paragraph 172.)

r.	c.	Mean vel. / Max. vel.	r.	c.	Mean vel. / Max. vel.	r.	c.	Mean vel. / Max. vel.
0·05	11·9	·320	2·65	67·8	·728	6·5	84·8	·770
0·10	16·7	·398	2·70	68·2	·729	6·6	85·0	·771
0·15	20·3	·446	2·75	68·6	·731	6·7	85·3	·771
0·20	23·4	·480	2·80	69·0	·732	6·8	85·5	·772
0·25	25·9	·506	2·85	69·3	·733	6·9	85·7	·772
0·30	28·3	·528	2·90	69·7	·734	7·0	86·0	·773
0·35	30·4	·545	2·95	70·0	·735	7·1	86·2	·773
0·40	32·3	·561	3·00	70·4	·736	7·2	86·4	·774
0·45	34·0	·574	3·05	70·7	·737	7·3	86·6	·774
0·50	35·7	·585	3·10	71·0	·737	7·4	86·8	·774
0·55	37·2	·595	3·15	71·4	·738	7·5	87·0	·775
0·60	38·7	·605	3·20	71·7	·739	7·6	87·2	·775
0·65	40·0	·613	3·25	72·0	·740	7·7	87·4	·776
0·70	41·3	·620	3·30	72·3	·741	7·8	87·6	·776
0·75	42·6	·627	3·35	72·6	·741	7·9	87·8	·776
0·80	43·7	·634	3·40	72·9	·742	8·0	88·0	·777
0·85	44·9	·639	3·45	73·2	·743	8·2	88·4	·778
0·90	45·9	·645	3·50	73·5	·744	8·4	88·7	·778
0·95	46·9	·650	3·55	73·7	·745	8·6	89·1	·779
1·00	47·9	·654	3·60	74·0	·745	8·8	89·4	·779
1·05	48·9	·659	3·65	74·3	·746	9·0	89·7	·780
1·10	49·8	·663	3·70	74·6	·747	9·2	90·0	·781
1·15	50·7	·667	3·75	74·8	·747	9·4	90·3	·781
1·20	51·5	·671	3·80	75·1	·748	9·6	90·6	·782
1·25	52·3	·674	3·85	75·3	·749	9·8	90·9	·782
1·30	53·1	·677	3·90	75·6	·749	10·0	91·2	·783
1·35	53·9	·680	3·95	75·8	·750	10·2	91·4	·783
1·40	54·6	·683	4·0	76·1	·750	10·4	91·7	·784
1·45	55·3	·686	4·1	76·5	·752	10·6	91·9	·784
1·50	56·0	·689	4·2	77·0	·753	10·8	92·2	·785
1·55	56·7	·691	4·3	77·5	·754	11·0	92·4	·785
1·60	57·4	·694	4·4	77·9	·755	11·2	92·6	·785
1·65	58·0	·696	4·5	78·3	·756	11·4	92·8	·786
1·70	58·6	·698	4·6	78·7	·757	11·6	93·1	·786
1·75	59·2	·701	4·7	79·1	·758	11·8	93·3	·787
1·80	59·8	·703	4·8	79·5	·759	12·0	93·4	·787
1·85	60·4	·705	4·9	79·9	·760	13·0	94·4	·789
1·90	60·9	·707	5·0	80·2	·760	14·0	95·2	·791
1·95	61·5	·708	5·1	80·6	·761	15·0	95·9	·793
2·00	62·0	·710	5·2	80·9	·762	16·0	96·6	·795
2·05	62·5	·712	5·3	81·3	·763	17·0	97·2	·797
2·10	63·0	·714	5·4	81·6	·763	18·0	97·7	·798
2·15	63·5	·715	5·5	81·9	·764	19·0	98·2	·798
2·20	64·0	·717	5·6	82·3	·765	20·0	98·6	·800
2·25	64·4	·718	5·7	82·6	·765	25·0	100·3	·804
2·30	64·9	·720	5·8	82·9	·766	30·0	101·5	·806
2·35	65·3	·721	5·9	83·2	·767	35·0	102·4	·808
2·40	65·8	·722	6·0	83·4	·767	40·0	103·1	·809
2·45	66·2	·723	6·1	83·7	·768	50·0	104·1	·809
2·50	66·6	·725	6·2	84·0	·768	70·0	105·2	·810
2·55	67·0	·726	6·3	84·3	·769	100·0	106·1	·810
2·60	67·4	·727	6·4	84·5	·770	Inf.	108·3	·810

[1] Extracted from Higham's "Hydraulic Tables," Nos. III. and IV.

APPENDIX 10.

TABLE OF THE DISCHARGES OF A WASTE-WEIR CHANNEL HAVING A BED-WIDTH OF 200 FEET AND A BED-SLOPE OF 1 IN 100.

(*Vide* Chapter III., paragraph 174.)

1	2	3	4	5	6	7	8	9
Total Depth.	Afflux Height, d_1.	Tail Depth, d_2.	Afflux Co-efficient, c_1.	Channel Co-efficient, c_2.	$\frac{D}{c_1 w \sqrt{2g}}$.	$\sqrt{d_1}(d_2 + \frac{1}{2} d_1)$.	Mean Velocity of Tail Channel.	Dis-charge, D.
Feet.	Feet.	Feet.					Feet per second.	Cubic feet per second.
1	0·30	0·70	0·60	41·00	0·495	0·495	3·40	476
2	0·74	1·26	0·60	52·10	1·510	1·505	5·78	1,457
3	1·24	1·76	0·60	59·00	2·85	2·87	7·78	2,741
4	1·70	2·30	0·62	64·4	4·47	4·46	9·66	4,444
5	2·15	2·85	0·64	68·8	6·34	6·29	11·42	6,510
6	2·63	3·37	0·66	72·0	8·29	8·29	13·03	8,784
7	3·08	3·92	0·68	75·0	10·46	10·45	14·55	11,407
8	3·52	4 48	0·70	77·5	12·80	12·84	16·04	14,374
9	3·95	5·05	072	79·5	15·23	15·28	17·41	17,585
10	4·36	5 64	0·74	81·4	17·87	17·87	18·80	21,210
11	4·77	6·23	0·76	83·1	20·56	20·51	20·11	25,057
12	5·17	6·83	0·78	84·5	23·34	23·34	21·38	29,203
13	5·54	7·46	0·80	85·8	26·24	26·20	22·57	33,668
14	6·00	co	0·80	86·8	29·44	29·40	23·61	37,775
15	6·47	8·53	0·80	87·7	32·65	32·61	24·56	41,893
16	6·94	9·06	0·80	88·6	36·03	36·00	25·51	46,236
17	7.42	9·58	0·80	89·3	39·47	39·52	26·43	50,645
18	7 86	10·14	0·80	90·0	43·10	43·06	27·28	55,304
19	8·33	10·67	0·80	90·7	46·88	46·81	28·19	60,157
20	8·80	11·20	0·80	91·3	50·67	50·63	29·03	65,035

In the Table the afflux coefficient c_1 has been taken approximately. Rankine[1] gives Poncelot and Lebros' coefficients, which were determined from experiments with orifices only about 8 inches wide, and it is believed none on a large scale have ever been made. The values of c_1 are increased up to a total flood-depth of 13 feet, on account of the more efficient discharging power of a deep channel. No further increase is thereafter made, as large waves, and consequently more friction, will be caused. Any errors made by these assumed values will be minimised in the calculations given in Appendices 12, 13 and 14, in the proportion that the discharges of the waste-weir cut bear to the total flood, and this will always be small.

[1] "Civil Engineering," 11th ed., art. 448, p. 680.

APPENDIX 11.

TABLE OF THE DISCHARGE FROM EACH FOOT OF LENGTH OF A CLEAR-OVERFALL WEIR, IN CUBIC FEET PER SECOND.

(*Vide* Chapter III., paragraph 176.)

Calculated by the formula:—$D = \frac{2}{3} cbd \sqrt{2gd}$.

Height of still water above weir crest = d.

CUBIC FEET PER SECOND.

Feet.	Decimals of a Foot.									
	·0	·1	·2	·3	·4	·5	·6	·7	·8	·9
0	0·00	0·10	0·28	0·52	0·81	1·14	1·51	1·91	2·34	2 80
1	3·29	3·81	4·35	4·91	5·49	6·09	6·71	7·35	8·01	8·69
2	9·39	10·11	10·85	11·61	12·39	13·19	14·01	14·85	15·71	16·58
3	17·47	18·37	19·29	20·22	21·17	22·13	23·11	24·10	25·11	26·13
4	27·16	28·21	29·27	30·35	31·44	32·55	33·67	34·80	35·94	37·09
5	38·25	39·42	40·60	41·80	43·01	44·23	45·46	46·71	47·97	49·24
6	50·52	51·81	53·11	54·42	55·74	57·07	58·41	59·76	61·12	62·44
7	63·87	65·26	66·66	68·07	69·49	70·92	72·36	73·81	75·27	76·64
8	78·22	79·71	81·21	82·72	84·24	85·77	87·31	88·86	90·42	91·98
9	93·55	95·13	96·72	98·32	99·93	101·55	103·18	104·82	106·47	108·12

NOTES.

1. The formula[1] $D = \frac{2}{3} cbd \sqrt{2gd}$, devised by Mr. James B. Francis, was deduced by him from an elaborate series of experiments at Lowell, U.S.A. (Lowell Hydraulic Experiments, New York, 1871) on a sharp-crested weir 10 feet long, of the full width of the stream, and with heads varying from 0·4 to 1·6 feet. Further experiments were made by Fteley, Stearns and others, from which Hamilton Smith has derived values for the co-efficient c for such weirs varying in length from 2 to 19 feet.

For a 2 feet weir with a depth of 0·10 foot, the co-efficient determined was 0·652; with a depth of 0·40 foot the co-efficient attained its minimum, 0·636, and then increased gradually to 0·648 with a depth of 1·00 foot, the maximum depth of the experiments with this length of weir.

For a 19 feet weir with a depth of 0·10 foot, the co-efficient was 0·657; with a depth of 0·70 foot the co-efficient attained its minimum, 0·618, and then increased gradually to 0·623 with a depth of 1·60 feet, the maximum depth of the experiments with this length of weir. The intermediate observations varied proportionally.

2. Generally in tables for the discharges of gauging weirs constructed on Francis' formula the value of the co-efficient c adopted is 0·512 throughout, although that

[1] *Vide* Table No. 42, p. 220 of "Public Water Supplies," by Turneaure and Russell. Chapman and Hall, 1901.

given in Molesworth's Pocket Book is calculated with the value of c as 0·67 throughout. Judging from the above experiments, from the analogy of Bazin's co-efficient (Appendix 9) and from general considerations, the co-efficient must increase slightly as the depth of the discharge increases (*vide* also the note to Appendix 10).

It is believed that, owing to the practical difficulty of gauging flows of great depth, experiments for determining their co-efficients have never been made.

3. The Table given above has therefore been constructed in the following manner. The co-efficients selected were :—

Depths in Feet.	Co-efficients.	Depths in feet.	Co-efficients.	Depths in Feet.	Co-efficients.	Depths in Feet.	Co-efficients.
0·00	0·600	2·50	0·623	5·00	0·635	7·50	0·646
0·50	0·605	3·00	0·625	5·50	0·638	8·00	0·647
1·00	0·610	3·50	0·628	6·00	0·640	8·50	0·649
1·50	0·615	4·00	0·630	6·50	0·642	9·00	0·650
2·00	0·620	4·50	0·633	7·00	0·644	9·50	0·650

The results thus obtained were afterwards slightly adjusted so as to get a series of discharges differing regularly from each other, and these were thereafter tabulated.

4. It must be remembered that the experiments quoted above were made on sharp-crested weirs, and therefore their results do not hold good for waste-weir walls as constructed in practice, which walls have crests of appreciable width.

Experiments[1] were made at the Cornell University Hydraulic Laboratory, U.S.A., by Messrs. Rafter and Williams, with weirs of different sections, 6·58 feet long and from 4·6 to 4·9 feet high, free access being provided for the air beneath the sheet of water. Unfortunately the sections experimented with were not similar to those which are adopted in India for waste-weir walls. The following are the co-efficients ascertained for weirs with vertical sides and horizontal crests, which are the ones nearest to the usual Indian section.

CO-EFFICIENTS.

Width of Weir Crest in Feet.	Height of still-water above Weir Crest in Feet.						
	2·0	2·5	3·0	3·5	4·0	4·5	5·0
2·62	0·53	0·54	0·57	0·59	0·62	0·65	0·69
6·56	—	0·46	0·47	0·47	0·48	0·49	0·50

The Table given in this Appendix will therefore be fairly exact for weirs having level crests with widths of from 3 feet to 4 feet and downstream batters of 1 in 4, (which will be the ones generally constructed in connection with storage reservoirs), and, especially, in the case of depths of water of from 5 feet to 6 feet, which will be usually the maximum flood depths on them.

[1] *Vide* Table No. 45, p. 224 of "Public Water Supplies," by Turneaure and Russell. Chapman and Hall, 1901.

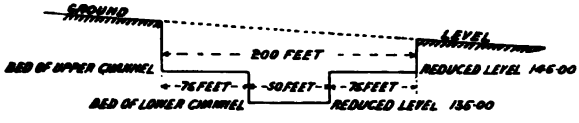
APPENDIX 12.

MÁLÁDEVI TANK PROJECT.

Temporary Waste-Weir Flood Calculations.—First Closure.—Table of Open Channel Discharges and Reservoir Flood-Absorption.

(*Vide* Chapter II., paragraph 137.)

Section of Open Channel.



Data.

The flood to commence when the channel is discharging at the rate of 5,300 cubic feet per second, or at the rate of about $\frac{1}{16}$th inch per hour run-off, which is a fair, small flood. To pass this discharge the 50-foot channel will have to flow 10 feet deep, so that the calculations will begin from Reduced Level 145·00. The rise of the reservoir surface level thereafter to be that due to a run-off at the rate of about 0·5 inch per hour from the catchment of 153 square miles.

In the first hour the reservoir will thus have to rise 4 feet; in the second hour, 3 feet; in the third hour, 2 feet; and, afterwards, 1 foot per hour, to deal with the assumed intensity of the flood.

The quantities discharged by the channel are calculated out foot by foot during the periods of rise.

Number of hours from Commencement of Flood.	At the end of each hour.			During the hour.				
	Reduced Level of Reservoir Surface.	Height of Reservoir Surface above Bed of Lower Channel.	Height of Reservoir Surface above Bed of Upper Channel.	Total Discharge of Lower Channel.	Total Discharge of Upper Channel.	Increment of Reservoir Storage (or Flood Absorption).	Total Flood dealt with. (Col. 5 + Col. 6 + Col. 7.)	Equivalent Run-off (of Col. 8) from Catchment.
1	2	3	4	5	6	7	8	9
		feet.	feet.	million cub. feet.	million cub. feet.	million cub. feet.	million cub. feet.	inches.
1	146·00	11	1	5·205	0·161	29·086		
	147·00	12	2	6·104	0·653	30·660	157·046	0·44
	148·00	13	3	7·073	1·417	32·281		
	149·00	14	4	8·037	2·425	33·944		
2	150·00	15	5	11·950	4·929	35·643		
	151·00	16	6	13·219	6·882	37·383	172·787	0·49
	152·00	17	7	14·532	9·085	39·164		
3	153·00	18	8	23·838	17·646	40·994		
	154·00	19	9	25·979	21·814	42·865	173·136	0·49
4	155·00	20	10	56·336	52·375	44·771	153·482	0·43
5	156·00	21	11	60·750	62·459	46·746	169·955	0·48
6	157·00	22	12	65·250	73·008	48·792	187·050	0·53
7	158·00	23	13	69·250	84·877	50·881	205·008	0·58
8	159·00	24	14	74·250	96·447	53·015	223·712	0·63
9	160·00	25	15	78·750	107·552	55·192	241·494	0·68
	Totals			520·523	541·730	621·417	1683·670	4·75
	Add total of Column 5				520·523			
	Total Discharge of Channels				1062·253			
	Percentage quantities				63·09	36·91	100·00	
	Add total High-Flood Discharge of Channels at Reduced Level 160·00 for 15 hours							8·19
	Total Run-off dealt with in 24 hours							12·94

The calculations thus show that with the open temporary waste-weir channel the Reservoir High-Flood Level will not rise above Reduced Level 160·00. At this level the discharge of the channel is 53,920 cubic feet per second, which is equal to a run-off of 0·546 inch per hour from the catchment of 153 square miles.

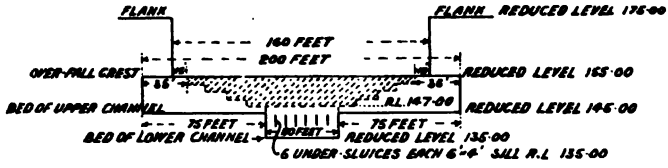
APPENDIX 13.

MÁLÁDEVI TANK PROJECT.

TEMPORARY WASTE-WEIR FLOOD CALCULATIONS—SECOND CLOSURE.—TABLE OF OUTLET HEADWALL DISCHARGES AND RESERVOIR FLOOD-ABSORPTION.

(*Vide* Chapter II., paragraph 138.)

SECTION OF OUTLET HEADWALL.



Data.

The flood to commence when the crest is flowing 4 feet deep, *i.e.*, is discharging 5,706 cubic feet per second, or at the rate of about $\frac{1}{17}$th inch per hour run-off, which is a fair, small flood.

The reservoir surface will therefore be at Reduced Level 159·00 at the commencement. The rise of the reservoir surface level thereafter to be that due to a run-off at the rate of about 0·5 inch per hour from the catchment of 153·00 square miles.

In the first and second hours, the Reservoir will thus have to rise 2 feet per hour, and, subsequently, 1 foot per hour, to deal with the assumed intensity of the flood.

The quantities discharged by the Headwall, &c., are calculated out foot by foot during the period of rise.

Note.

In construction the Headwall crest should be finished off with a central gap—as shown by dotted lines—so as to give it a greater discharging power, to deal with the flood earlier, and to lessen the action of the flood on the foundations.

Number of hours from Commencement of Flood.	At the end of each hour.		During the hour.				
	Reduced Level of Reservoir Surface.	Height of Reservoir Surface above Headwall Crest.	Total Discharge of Headwall Crest.	Total Discharge of Under-sluices.	Increment of Reservoir Storage (or Flood-Absorption).	Total Flood dealt with. (Col. 4 + Col. 5 + Col. 6.)	Equivalent Run-off (of Column 7) from Catchment.
1	2	3	4	5	6	7	8
		Feet.	Million Cubic Feet.	Million Cubic Feet.	Million Cubic Feet.	Million Cubic Feet.	Inches.
1	160·00	5	12·311	6·635	55·192	155·197	0·44
	161·00	6	17·011	6·635	57·413		
2	162·00	7	21·324	6·635	59·711	182·708	0·51
	163·00	8	26·417	6·635	61·986		
3	164·00	9	63·725	13·478	64·338	141·541	0·40
4	165·00	10	75·308	13·687	66·734	155·729	0·44
5	166·00	11	87·725	13·687	69·391	170·803	0·48
6	167·00	12	100·310	13·892	72·329	186·531	0·52
7	168.00	13	113·650	14·101	75·336	203·087	0·57
8	169·00	14	127·530	14·301	78·396	220·236	0·62
Totals			645·311	109·695	660·826	1,415·832	3·98
Add total of Column 4				645·311			
Total Outlet Headwall Discharge				755·006			
Percentage quantities				53·32	46·68	100·000	
Add total Outlet Headwall Discharge for 16 hours							6·71
Total Run-off dealt with in 24 hours							10·69

By calculation it is ascertained that the discharge of the tail channels when flowing level with the crest of the Headwall (Reduced Level 155·00) will be about 80,000 cubic feet per second. As the maximum discharge to be dealt with is only 49,367 cubic feet per second, (due to a run-off of 0·5 inch from the catchment), the Headwall will thus discharge throughout as a Clear Overfall Weir.

To determine the discharge of the Under-sluices the approximate effective head on them is first calculated, *i.e.*, the difference between the Reservoir surface upstream and that of the Tail Channel downstream. This latter is deduced from Appendix 10.

The calculations thus show that with the Outlet Headwall crest constructed as shown in full lines (much more so when built as shown by dotted lines), the Reservoir High-Flood Level will not rise above Reduced Level 169·00. At this level the discharge of the Headwall and Under-sluices is 41,424 cubic feet per second, which is equal to a run-off of 0·42 inch per hour from the catchment of 153 square miles.

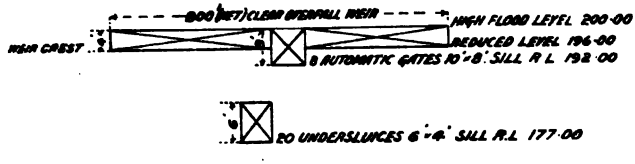
APPENDIX 14.

MÁLÁDEVI TANK PROJECT.

PERMANENT WASTE-WEIR FLOOD CALCULATIONS.—TABLE OF WASTE-WEIR DISCHARGES AND RESERVOIR FLOOD-ABSORPTION.

(*Vide* Chapter III., paragraph 193.)

FLOOD SECTION OF WEIR.



Data.

The flood to commence when the Reservoir level has risen 10 feet above the sills of the under-sluices, *i.e.*, to Reduced Level 187·00. Its initial amount will thus be 7,600 cubic feet per second, or at the rate of about $\frac{1}{15}$th inch per hour run-off, which is a fair, small flood.

All sluices (except the automatic gates) to be fully open and the weir crest unobstructed. The Reservoir surface to rise 1 foot per hour.

These calculations do not take into account the discharge from the Outlet sluices, which will vary from 6,221 to 7,027 cubic feet per second, or from 0·06 to 0·07 inch per hour run-off, during the period considered.

Number of hours from Commencement of Flood.	At the end of each hour.		During the hour.				
	Reduced Level of Reservoir Surface.	Height of Reservoir Surface above Bed of Waste-Weir Tail Channel.	Total Discharge of Under-sluices.	Total Discharge of Waste-Weir Crest.	Increment of Reservoir Storage (or Flood-Absorption).	Total Flood dealt with. (Col. 4 + Col. 5 + Col. 6.)	Equivalent Run-off (of Column 7) from Catchment.
1	2	3	4	5	6	7	8
		Feet.	Million Cubic Feet.	Million Cubic Feet.	Million Cubic Feet.	Million Cubic Feet.	Inches.
1	188·00	11	28·440	—	128·438	156·878	0·44
2	189·00	12	30.420	—	130·886	161·306	0·45
3	190·00	13	32·220	—	133·357	165·577	0·47
4	191·00	14	33·840	—	135·747	169·587	0·48
5	192·00	15	35·280	—	138·028	173·308	0·49
6	193·00	16	36·720	—	140·328	177·048	0·50
7	194·00	17	38·160	—	142·646	180·806	0·51
8	195·00	18	39·600	—	144·985	184·585	0·52
9	196·00	[1] 19	41·040	—	147·356	188·396	0·53
10	197·00	20	42·480	5·137	149·733	197·350	0·55
11	198·00	21	43·200	19·656	152·128	214·984	0·60
12	199·00	22	43·200	41·209	154·555	238·964	0·67
13	200·00	[2] 23	43·200	67·777	157·002	267·979	0·75
Totals			487·800	133·779	1,855·189	2,476·768	6·96
Add total of Column 4				487·800			
Total Waste-Weir Discharge				621·579			
Percentage quantities				25·10	74·90	100·00	
Add total Waste-Weir High-Flood Level Discharge for 11 hours (including automatic gates)							4·60
Total Run-off dealt with in 24 hours							11·56

[1] Masonry crest of Waste-Weir.
[2] At Reduced Level 200·00 the automatic gates come into action and the total Waste-Weir discharge becomes 41,284 cubic feet per second, or, at the rate of 0·418 inch per hour run-off from the catchment of 153 square miles.

To determine the discharge of the Under-sluices, the approximate effective head on them is first calculated, *i.e.*, the difference between the Reservoir surface upstream and that of the Tail Channel downstream. This latter is deduced from Appendix 10.

At first the sluices have partly drowned and partly clear openings; and finally, wholly drowned openings.

APPENDIX 15.

RECAPITULATION OF MÁLÁDEVI TANK PROJECT.

(*Vide* Chapter V., paragraph 244.)

1	2	3
Items.	Amount. Rs.	Total. Rs.
I. WORKS.		
1. *Headworks.*		
Preliminary expenses—survey, gauging, &c.	Previous expenditure to be written off.	
2. *Land.*		
Compensation for acquisition		75,000
3. *Masonry Works.*		
Outlet	1,22,372	
Waste-Weir	1,56,007	2,78,379
4. *Buildings.*		
Bungalow and outhouses	6,727	
Overseers' quarters	1,121	
Watchmen's quarters	1,052	
Temporary buildings	1,100	10,000
5. *Earthwork.*		
Dam embankment, pitching, puddle trench, &c. ...		8,46,954
6. *Plantation.*		
Planting and preserving trees around the Reservoir ...		5,000
7. *Miscellaneous.*		
Maintenance during construction	10,000	
Boat and boat-house	1,000	11,000
Total I. Works		**12,26,333**
II. ESTABLISHMENT.		
Construction Establishment (at 18 per cent. on the cost of the project exclusive of land charges—*i.e.*, on Rs.11,51,333)	2,07,240	
Direction and accounts at 5 per cent. on ditto, ditto	57,560	
Total II. Establishment ...		**2,64,800**
III. TOOLS AND PLANT.		
Tools and Plant (at 5[1] per cent. on I. WORKS (less land charges) say:—		**58,000**
A. ***Total Direct Charges*** ...		**15,49,333**
IV. SUSPENSE (NOT AFFECTING CHARGES TO GRANT).		
Capitalisation of abatement of Land Revenue	75,000	
Leave and pension allowances (at 14 per cent. on II. ESTABLISHMENT)	37,100	
Interest on I. WORKS at 2 per cent. on year's expenditure and 4 per cent. on previous expenditure[2] ...	1,22,600	
B. ***Total Indirect Charges*** ...	...	**2,34,700**
Grand Total, Direct and Indirect Charges ...		**17,84,033**

[1] An extra allowance beyond the usual one of 1½ per cent. has been made to provide for the large amount of plant necessary.

[2] The work is estimated to take 5 years for completion; hence the interest charges amount to 2 per cent. for 5 years on the whole sum at charge.

APPENDIX 16.

ABSTRACT ESTIMATE OF MÁLÁDEVI TANK PROJECT.

(*Vide* Chapter V., paragraph 244.)

1	2	3	4	5	6	7
Quantity.	Unit.	Items.	Rate.	Per.	Amount.	Remarks.
			Rs. A. P.		Rs.	
		I. DAM EMBANKMENT.				
		1. *Puddle Trench.*				
1,140,841	Cft.	Excavation in soil[1] ...	1 8 0	100	17,113	
182,495	,,	Ditto in murum[1] ...	1 8 0	,,	2,737	
23,803	,,	Ditto in rock[1]	4 0 0	,,	952	
27,495	,,	Ditto rock in puddle trench drain[1]	6 0 0	,,	1,650	
106,675	,,	Ditto nulla puddle trench[1]	0 12 0	,,	800	
1,347,139	,,	Filling dam puddle trench	1 4 0	,,	16,839	
213,350	,,	Ditto nulla ,, ,,	1 0 0	,,	2,134	
4,219	Rft.	Puddle trench drain, excavation and filling ...	1 0 0	Rft.	4,219	
					46,444	
		2. *Central Wall.*				
18,361	Cft.	Excavation in rock[1] ...	3 0 0	100	551	
50,386	,,	Concrete	15 0 0	,,	7,558	
17,358	,,	Masonry facing	32 0 0	,,	5,555	
					13,664	
		3. *Drainage Works.*				
		(*a*) *Foundation, Surface, and Cross Drains.*				
610,012	Cft.	Excavation in soil[1] ...	0 8 0	100	3,050	
235,860	,,	Filling with clay ...	0 12 0	,,	1,769	
188,280	,,	Ditto gravel	2 0 0	,,	3,766	
			Total (*a*) ...		8,585	

[1] In all estimates the rates for excavation are reduced by the value of the material to constructional items (dam, drystone, toe, &c.), where full rates are allowed.

TABLE—*continued.*

1	2	3	4	5	6	7
Quantity.	Unit.	Items.	Rate.	Per.	Amount.	Remarks.
		I. DAM EMBANKMENT.—*continued.*	Rs. A. P.		Rs.	
		(b) Downstream Drain.				
492,844	Cft.	Excavation in soil ...	0 12 0	100	3,896	
492,844	,,	Filling	2 0 0	,,	9,857	
210	Rft.	Rear drain below dam ...	1 0 0	Rft.	210	
			Total (b) ...		13,963	
			Total (a) + *(b)*		22,548	
		4. *Drystone Toe.*				
51,331	Cft.	Excavation in rock[1] ...	3 0 0	100	1,540	
68,533	,,	Excavation in soil[1] ...	0 4 0	,,	171	
553,383	,,	Concrete	15 0 0	,,	83,007	
4,161,473	,,	Drystone	4 0 0	,,	1,66,459	
					2,51,177	
		5. *Dam Embankment.*				
26,836,904	Cft.	Embankment	1 8 0	100	4,02,553	
34,822	Sft.	Pitching 12″ thick ...	6 0 0	,,	2,089	
187,081	,,	Ditto 12″ to 18″ thick ...	8 0 0	,,	14,966	
191,855	,,	Ditto 18″ thick	10 0 0	,,	19,186	
20,935	,,	Ditto 24″ thick	15 0 0	,,	3,140	
					4,41,934	
		6. *Masonry Wall at Top.*				
76,657	Cft.	Coursed rubble masonry	22 0 0	100	16,865	
36,516	,,	Concrete	16 0 0	,,	5,843	
4,296	Rft.	Coping, extra	0 8 0	Rft.	2,148	
					24,856	
		7. *Miscellaneous.*				
		Road to bungalow from Rambhori	Lump	...	2,000	
		Works' roads	Ditto	...	2,000	
		Sundries	Ditto	...	2,000	
					6,000	
		Total Rs.	...	...	8,06,623	
		Contingencies at 5 per cent.	...	...	40,331	
		Grand Total—I. Dam Embankment.		Rs.	**8,46,954**	

[1] See note on p. 289.

TABLE—*continued.*

1	2	3	4	5	6	7
Quantity.	Unit.	Items.	Rate.	Per.	Amount.	Remarks.
			Rs. A. P.		Rs.	
		II. OUTLET.[1]				
297,737	Cft.	Earth excavation[2] ...	0 6 0	100	1,116	
31,703	,,	Rock excavation[2] ...	2 0 0	,,	634	
106,800	,,	Puddle filling	1 8 0	,,	1,602	
292,562	,,	Concrete	15 0 0	,,	43,884	
87,683	,,	Masonry facing	32 0 0	,,	28,059	
2,569	,,	Block-in-course masonry	50 0 0	,,	1,285	
3,193	,,	Arch work	55 0 0	,,	1,756	
2,235	,,	Coping	50 0 0	,,	1,117	
387	,,	Cornice	60 0 0	,,	232	
84,303	,,	Drystone toe	4 0 0	,,	3,372	
6	No.	Sluice Gates 6′ × 4′ ...	2,000 0 0	Each	12,000	
2,182,500	Cft.	Channel excavation[2] ...	0 6 0	100	8,184	
569,637	,,	Dam embankment ...	1 8 0	,,	8,545	
10,968	Sft.	Pitching	6 0 0	,,	658	
1,360	Cft.	Masonry of top wall ...	24 0 0	,,	326	
2	No.	Turbine sluices	700 0 0	Each	1,400	
380	Cwt.	Turbine pipes	6 4 0	Cwt.	2,375	
		Total Rs.	...	...	1,16,545	
		Add contingencies at 5 *per cent.*	...	...	5,827	
		Grand Total—II. Outlet. ...		Rs.	**1,22,372**	
		III. WASTE-WEIR.				
		A. Clear Overfall Section.				
71,197	Cft.	Excavation in soil and murum[2]	0 12 0	100	534	
37,895	,,	Concrete	15 0 0	,,	5,684	
22,729	,,	Masonry facing	32 0 0	,,	7,273	
3,275	,,	Block-in-course masonry	50 0 0	,,	1,638	
1,493	Rft.	Cornice	1 0 0	Rft.	1,493	
		Total Rs.	...	...	16,622	
		Add contingencies at 5 *per cent.*	...	...	831	
		Grand Total Rs. ...	...	...	17,453	

[1] This estimate provides for all work in the 200 feet length occupied by the temporary Waste-Weir Channel.

[2] See note on p. 289.

TABLE—*continued.*

1	2	3	4			5	6	7
Quantity.	Unit.	Items.	Rate.			Per.	Amount.	Remarks.
		III. WASTE-WEIR.—*con.*	Rs.	A.	P.		Rs.	
		B. Under-Sluice Section.						
42,136	Cft.	Excavation in murum[1] ...	0	8	0	100	211	
37,929	,,	Ditto in soft rock[1] ...	1	8	0	,,	569	
32,780	,,	Ditto in hard rock[1] ...	4	0	0	,,	1,311	
57,188	,,	Concrete hearting ...	15	0	0	,,	8,578	
17,192	,,	Masonry facing	32	0	0	,,	5,501	
2,205	,,	Block-in-course masonry	50	0	0	,,	1,102	
16,348	,,	Ashlar masonry	60	0	0	,,	9,809	
2,821	,,	Ditto arching	70	0	0	,,	1,975	
263	,,	Corbels	60	0	0	,,	158	
80	No.	Sluice sills and lintels ...	8	0	0	Each	640	
562	Rft.	Cornice	1	0	0	Rft.	562	
522	Cwt.	Iron rails...	4	0	0	Cwt.	2,088	
130	Cft.	Cut teak woodwork ...	4	0	0	Cft.	520	
20	No.	Sluice gates, 6′ × 4′ ...	1,200	0	0	Each	24,000	
		Total Rs.	...			...	57,024	
		Add contingencies at 5 per cent.	...			...	2,851	
		Grand Total Rs. ...	...			...	59,875	
		C. Automatic Gate Section.						
18,736	Cft.	Excavation in murum[1] ...	0	8	0	100	94	
5,350	,,	Ditto in soft rock[1] ...	1	8	0	,,	80	
5,787	,,	Ditto in hard rock[1] ...	4	0	0	,,	231	
15,367	,,	Concrete	15	0	0	,,	2,305	
5,925	,,	Masonry facing	32	0	0	,,	1,896	
3,949	,,	Block-in-course masonry	50	0	0	,,	1,960	
706	,,	Arching	55	0	0	,,	388	
144	Rft.	Cornice	1	0	0	Rft.	144	
8	No.	Automatic gates, 10′ × 8′	1,800	0	0	Each	14,400	
		Total Rs.	...			...	21,498	
		Add contingencies at 5 per cent.	...			...	1,075	
		Grand Total Rs. ...	...			...	22,573	
		D. Arcade Footpath.						
3,269	Cft.	Concrete	15	0	0	100	490	
17,179	,,	Block-in-course masonry	32	0	0	,,	5,497	
5,223	,,	Arching	55	0	0	,,	2,873	
1,020	,,	Corbels	60	0	0	,,	612	
160	No.	Cut-and ease-water caps	5	0	0	Each	800	
1,035	Cft.	Cut teak woodwork ...	4	0	0	Cft.	4,140	
2	No.	Travelling winches ...	250	0	0	Each	500	
		Tramway...	Lump			...	1,500	
		Total Rs.	...			...	16,412	
		Add contingencies at 5 per cent.	...			...	821	
		Grand Total Rs. ...	...			...	17,233	

[1] See note on p. 289.

TABLE—*continued.*

1	2	3	4	5	6	7
Quantity.	Unit.	Items.	Rate.	Per.	Amount.	Remarks.
		III. WASTE-WEIR.—*con.*	Rs. A. P.		Rs.	
		E. Protective Works.				
82,094	Cft.	Excavation in soil[1] ...	0 8 0	100	410	
17,671	,,	Ditto in soft rock[1] ...	1 8 0	,,	265	
12,162	,,	Ditto in hard rock[1] ...	5 0 0	,,	608	
38,372	,,	Concrete	15 0 0	,,	5,757	
20,522	,,	Coursed rubble masonry, 1st sort...	32 0 0	,,	6,567	
27,913	,,	Ditto, 2nd sort	24 0 0	,,	6,699	
		Protective bank near Rambhori	Lump	...	800	
		Total Rs.	...	...	21,106	
		Add contingencies at 5 per cent.	...	...	1,055	
		Grand Total Rs. ...	...	...	22,161	
		F. Approach and Tail Channels.				
3,916,853	Cft.	Excavation in soil[1] ...	0 6 0	100	14,688	
327,600	,,	Embankment	0 6 0	,,	1,228	
		Total Rs.	...	...	15,916	
		Add contingencies at 5 per cent.	...	...	796	
		Grand Total Rs. ...	...	...	16,712	
		Grand Total—III. Waste-Weir. ...		Rs.	**1,86,007**	
		IV. BUILDINGS.				
		Bungalow	...	...	4,893	
		Out-houses	...	...	1,834	
		Overseers' Quarters ...	...	...	1,121	
		Watchmen's Quarters ...	...	...	1,052	
		Temporary Buildings ...	...	...	1,100	
		Grand Total—IV. Buildings. ...		Rs.	**10,000**	

[1] See note on p. 289.

APPENDIX 17.

MÁLÁDEVI TANK PROJECT.

TABLE OF RESERVOIR CONTENTS AT EACH CONTOUR.

(*Vide* Chapter I., paragraph 21.)

1	2	3	4	5	6
Reduced level of Contour.	Square root of area of Contour.	Area of Contour.	Contents between successive Contours.	Contents up to each Contour.	Remarks.
		Square feet.	Cubic feet.	Cubic feet.	
135	**4,328**	**18,731,584**	—	—	**Outlet sill.**
136	4,427	19,598,320	19,163,320	19,163,320	
137	4,527	20,493,729	20,044,359	39,207,679	
138	4,626	21,399,876	20,945,169	60,152,848	
139	4,725	22,325,625	21,861,117	82,013,965	
140	4,825	23,280,625	22,801,458	104,815,423	
141	4,924	24,245,776	23,761,567	128,576,990	
142	5,023	25,230,529	24,736,519	153,313,509	
143	5,122	26,234,884	25,731,073	179,044,582	
144	5,222	27,260,284	26,747,417	205,791,999	
145	**5,321**	**28,313,041**	**27,786,529**	**233,578,528**	
146	5,465	29,866,225	29,086,177	262,664,705	
147	5,609	31,460,881	30,660,097	293,324,802	
148	5,754	33,108,516	32,281,194	325,605,996	
149	5,898	34,786,404	33,944,004	359,550,000	
150	6,042	36,505,764	35,642,628	395,192,628	
151	6,186	38,266,596	37,382,724	432,575,352	
152	6,330	40,068,900	39,164,292	471,739,644	
153	6,475	41,925,625	40,993,758	512,733,402	
154	6,619	43,811,161	42,864,937	555,598,339	
155	**6,763**	**45,738,169**	**44,771,209**	**600,369,548**	
*	*	*	*	*	
177	10,203	104,101,209	103,013,304	2,202,486,148	Sill of waste-weir under-sluices.
*	*	*	*	*	
196	12,188	148,547,344	147,356,121	4,498,935,205	Masonry crest of waste-weir.
*	*	*	*	*	
200	12,579	158,231,241	157,001,700	5,112,353,401	H.F.L. and F.S.L. Crest of teak shutters.

NOTES.

1. Surveyed contours, &c., are shown in heavy type figures.
2. Interpolated contours, &c., are shown in light type figures.
3. The table is constructed on the assumption that the square roots of the areas of the interpolated contours between the surveyed ones are in arithmetical progression.

APPENDIX 18.

SPECIFICATIONS FOR THE PUDDLE TRENCH, DAM EMBANKMENT AND PITCHING.

(*Vide* Chapter V., paragraph 245.)

INDEX TO CLAUSES.

	References to Chapter II. Para.	Page
1. General	—	—
I. The Puddle Trench.		
2. Position and Extent	85—89	82—86
3. Depth	86	83
4. Side-slopes	88	86
5. Bottom-width	87	85
6. Key Trench	94	92
7. River Crossing: Concrete Wall	95	92
8. Minor Drainage Crossings	95	93
9. Springs in the Trench	92	90
10. Material for Filling	92	89
11. Method of Filling	88, 92	86, 88
12. Puddle Trench Drain	97	94
II. The Foundation of the Dam.		
13. Clearance of Site	113	116
14. Foundation Benches and Trenches	112, 113	114, 116
15. The Surface Drain	107	105
16. The Downstream Drain	107	106
17. Rear Drains	108	108
18. General	—	—
III. The Superstructure of the Dam.		
19. The Sections of the Dam	65	63
20. The Material of the Dam	79—81	76—80
21. Mixing and Spreading	118	119
22. Consolidation	119, 122	120, 123
23. Watering	117	118
24. Thickness of Layers	120	122
25. Slopes of Layers	121	123

III. The Superstructure of the Dam—*continued.*	References to Chapter II. Para.	Page
26. Testing the Construction of the Dam . . .	125	126
27. Junctions of Earthwork	123	124
28. Finishing off the Embankment	124	125
29. Excavations for Materials	102	101
IV. Pitching.		
30. The Extent and Thickness of the Pitching . .	154	152
31. The Stone	—	—
32. Laying	153	151
33. Foundation and Top Courses	155	153

SPECIFICATIONS FOR EARTHEN DAMS.

1. General.—The dam is to consist of an earthen embankment with puddle and concrete trenches, as shown on the drawings. It is to be faced with pitching on the water-slope and to be provided with an outlet (s) and waste-weir (s). All works in connection with it shall be executed in strict accordance with the plans and estimates except where written orders by the Engineer permit deviations from them to be made. Only the best materials and the soundest form of construction shall be permitted, and everything shall be executed in a thoroughly workmanlike way.

The following specifications proceed in the order in which the works will be carried out :—

I. The Puddle Trench.

2. Position and Extent.—The puddle trench shall be excavated continuously along the centre line of the dam and shall extend from the high-flood margin on one bank to that on the other bank.

3. Depth.—The trench shall be excavated until sound rock, or impervious soil, is met with, and shall be carried into it for a minimum depth of 1 foot. It shall not be filled until it has been passed by the Engineer himself, who shall carefully consider, by taking into account the nature of the subsoils passed through and the full-supply pressure head of the reservoir, whether the depth is sufficient to ensure the staunchness of the work. As a general guide it may be stated that, where sound rock is not met with but compact subsoils exist, the depth of the trench shall not be less than half the full-supply pressure head of the reservoir at the point considered, and, where the subsoils are less compact, the depth shall not be less than that pressure head.

In both these cases the bottom of the puddle trench shall be carried for at least one-tenth of that pressure head into sound, retentive clayey material.

The puddle trench shall not be continued below the foundation of the outlet culvert, but, where that is crossed at a lower level, shall be replaced by a concrete trench well keyed into the puddle at its flanks.

The puddle trench shall be replaced by a concrete trench when the pressure head is great and the sub-strata are of fissured, hard material, as the Engineer by written order may direct. The concrete shall be laid and consolidated as specified in clause 6.

4. Side-slopes.—The ratio of the side-slopes shall be determined by the Engineer from a consideration of the nature of the soils through which the trench passes, but in no case shall it be less than 1 in 8 in rock, nor less than 1 in 4 in soils, nor greater than is necessary to ensure the stability of the sides of the trench until it is filled in. The side-slopes shall be steepest at the base and flattest at ground level, and nowhere shall they be taken out vertically. Shoring shall not be resorted to if it can be possibly avoided, but, where it is necessary, it shall be made in short lengths with slopes battering not less than 1 in 8, and the trench shall be rapidly filled in.

5. Bottom-width.—The bottom-width of the trench shall not be less than one-eighth of the full-supply pressure head *plus* 3 feet, nor less than 6 feet. It shall be taken out level in cross section, except where a key trench shall be ordered (clause 6), and shall not have any vertical steps in longitudinal section, all changes of level being effected by gentle slopes.

6. Key Trench.—If there is fissured rock at the bed of the puddle trench, a key trench, not less than 3 feet wide, shall be excavated along the centre line of the dam, and shall extend, both vertically and horizontally, into sound rock. It shall be filled with fine, rich concrete in well-rammed 4-inch layers constructed at once, one on top of the other, until the whole key trench is completed, and on this filling a concrete wall shall be carried up at least 2 feet above the bed of the main trench, and shall be finished off with battering sides and a smooth, rounded top.

7. River Crossing—Concrete Wall.—At the river crossing the puddle trench shall be replaced by a concrete wall whose foundations shall be carried at least 2 feet into perfectly sound, unfissured rock, and whose superstructure shall be raised to a height not less than one-eighth of the full-supply pressure head. The side-slopes of the superstructure shall be faced with masonry, and shall batter not less than one-eighth; its top-width shall not be less than 6 feet, and shall be

finished off with a longitudinal key recess to secure proper union with the embankment. The concrete wall shall be continued well into the natural banks of the river, and shall be well keyed into the puddle trench, which shall be thickened out to overlap it on both sides for a distance not less than one-quarter of the full-supply pressure head. The concrete wall shall be constructed with smooth sides, plastered with mortar or luted with clay, and small vertical pilasters and recesses shall be formed on its sides so as to make it key thoroughly with the earthwork of the dam. On the downstream side of this wall shall be constructed a drain leading to the rear drain (clause 17).

8. Minor Drainage Crossings.—At these the puddle trench shall be deepened and widened, or shall have a key trench constructed at its base, or shall be replaced by a concrete wall, as the Engineer shall direct in writing after taking into consideration the depth of the full-supply pressure head and the nature of the subsoils.

9. Springs in the Trench.—All large springs in the trench shall be led in pipes to its downstream side, and shall be carried up with the trench filling. When the trench filling has reached a sufficient height, the pipes shall be carefully plugged with fine cement concrete. Small quantities of water shall be carefully drained off so as not to affect the filling of the trench in any way.

10. Material for Filling.—The material to be employed for filling shall be the most retentive clay procurable within one-half a mile of the site: it shall be quite free from vegetable or slushy matter and earth impregnated with salt, and shall not be liable to deterioration by the infiltration of water. It shall not be mixed with gritty soils or with sand.

11. Method of Filling.—Just before filling is commenced, the sides and bottom of the trench shall be slightly roughened, so that the new material may unite perfectly with the natural soil. Where the bed of the trench is in rock or other hard material, it shall be swept and washed free from dust, etc. The bed having been thus prepared, shall be wetted and the first layer of the filling material, which shall have been made into stiff plastic balls, shall be thrown on to the bed and trodden so as to secure a perfect junction with it. The rest of the filling shall be constructed in layers not exceeding 3 inches in thickness, and these shall be smoothened and then thoroughly consolidated by ramming or by rolling with heavy rollers, and, after their completion, they shall be wetted slightly to receive and unite with the next layer. The material of the filling shall be only slightly damp, and all clods in it shall be broken up. The surface of the layers shall rise down-stream at an inclination of about 1 in 6. At vertical and horizontal

intervals and breaking joint with each other shall be excavated wedge-shaped trenches, 3 feet wide at top and 1 foot deep, and these shall be refilled with the clayey material and thoroughly rammed before the upper layers are constructed.

Where sand pockets occur at the upstream side of the trench, they shall be excavated out just before the filling reaches them, and the filling shall be thoroughly worked into the cavities thus formed. If much sand is met with, the trench shall be widened on the upstream side to such an extent as the Engineer may direct.

The filling shall be continued until the trench is completely filled, and for 2 feet above ground surface, this upper part being constructed together with the embankment on each side, and thereafter the embankment shall be carried uniformly over it. If the construction of the embankment is not at once to be undertaken, the puddle shall be covered over with 2 feet of soil to prevent it from cracking—for the same reason the layers of the puddle trench filling shall be constructed, one on top of the other, as rapidly as possible.

12. Puddle Trench Drain.—When such a drain has been designed, it shall be carefully constructed of the specified dimensions in a trench excavated in the main bed of the puddle trench and founded throughout on an unyielding foundation, and shall have a continuous longitudinal slope to its outfall, which shall be continued across all depressions in the puddle trench. The walls and covering slabs shall be constructed of as long stones as possible, laid in mortar, with half-inch dry joints at intervals of 5 feet intermediate with each other at the bed and sides. The whole shall be surrounded with dry material (in accordance with the detailed drawing), and this shall be carefully covered with the material of the trench filling for a depth not exceeding 3 feet. After the whole drain has been completed, it shall be tested with water to see that it is perfectly free from obstructions, and, when this has been proved to be the case, the filling of the main trench shall be resumed and completed. Should the flow be obstructed, the defective part shall be cut out and properly remade.

Wherever the nature of the ground permits, the drain shall be led out of the dam and allowed to discharge into the natural drainage channels, and shall in this case be constructed in sections independent of each other, and each starting at least 5 feet downstream of the end of the upper section.

II. The Foundation of the Dam.

13. Clearance of Site.—The whole site to be covered by the embankment shall be cleared for its full width of all rubbish, loose stones,

powdery or cracked earth, silt, sand, and all soils charged with salts or in other respects unsuitable for the foundation. All trees, shrubs and vegetation growing on the site shall be completely rooted out, and the holes thus formed shall have their sides neatly sloped.

Particular care shall be taken thoroughly to clear the site where the dam crosses the main and tributary streams, water-courses, &c., so as to obtain a perfectly sound foundation at these places.

The material thus removed shall not be utilised for the construction of the dam without the written permission of the Engineer, and, if not utilised, shall be neatly spread or disposed of as may be directed.

The foundation of the dam shall consist of sound material not liable to deteriorate under the action of water, and shall be able to support the dam while sustaining only a small and uniform amount of compression.

14. Foundation Benches and Trenches.—The whole site outside the puddle trench shall be excavated into foundation benches parallel to the centre line of the dam, with a width from crest to crest of 20 feet and a depth of trough of 3 feet, unless otherwise directed in writing or shown on the drawings. The side next the centre line of the dam shall be excavated with a slope parallel to that of the dam on that side, and the slope of the other side of the bench shall be adjusted accordingly.

At the base of the foundation benches shall be excavated a series of trenches with slides sloping at 1 in 4, 3 feet wide at bottom and 3 feet deep, with a central one on each side 4 feet wide at bottom and 5 feet deep. On the upstream side these shall be filled with the most retentive clay procurable within half a mile of the site, and this shall be thoroughly well rammed. On the downstream side the trenches shall be filled with quarry spauls, small rubble, &c., covered with fine dry material, to form drains, which shall have a continuous fall throughout their course, and shall be led, at intervals of about 300 feet, out of the dam by cross drains of similar section and construction into the downstream drain (clause 16). The lower drains shall not be begun nearer than 5 feet from the cross drains of the upper ones.

Where the dam rests on steep side-long ground, the foundation benches shall be excavated at right angles to the axis of the embankment, and with their main slope rising towards the surface of the ground. On the upstream side the benches shall be divided into sections not more than 50 feet long, each separated from the others by unexcavated strips 5 feet wide running parallel to the centre line of the dam, and, on the reservoir side of these shall be stop trenches filled with carefully rammed clay. On the downstream side of the puddle trench the foundation

trenches shall be excavated at right angles to the dam in the furrows of the benches, and shall be led straight out of the dam, independently of each other.

15. The Surface Drain.—The natural ground below the downstream toe of the dam shall be dressed off for a width of 30 feet, with an inclination from the dam of 1 in 10 to form a surface drain, which shall be constructed in disconnected sections corresponding with the natural slope of the ground and not exceeding 300 feet in length, and having a longitudinal fall sufficient to carry off the drainage to be dealt with. Each section shall be separated from the neighbouring ones by an unexcavated strip not less than 10 feet wide so as to prevent the formation of longitudinal scour channels. The discharge of these sections shall be led by outfall gutters, leading from just upstream of the unexcavated strips, and shall be led across the downstream drain (clause 16) in watertight channels and shall be continued as open gutters to discharge clear of it.

16. The Downstream Drain.—Just clear and downstream of the surface drain shall be a downstream drain, which shall have a base width of 5 feet, side-slopes not flatter than 1 in 4, and a continuous longitudinal fall. It shall, if possible, be carried 1 foot into sound rock, but, where this does not exist, it shall have a depth of not less than 10 feet. Wherever practicable, it shall be constructed in discontinuous sections, discharging by cross drains of similar design into the natural drainage lines, and each of these sections shall be separated from the neighbouring ones by unexcavated strips 10 feet wide.

At the base of this trench shall be a drystone slab drain 4 inches wide and 6 inches deep at the head, and gradually increasing in section as it is continued downstream. The trench shall be filled with sound, dry material from the excavations, &c., up to within 3 feet of the ground surface, (the larger particles being placed at the bottom), and this shall be lined at the sides for 1 foot and covered at the top by 2 feet of fine gravel and coarse sand. The whole shall be finished off by a cover of soil carried 1 foot above ground level.

17. Rear Drains.—At all valley lines crossed by the dam shall be rear drains leading from the puddle trench drain at right angles to the axis of the dam, with as steep longitudinal slopes as practicable, into the natural drainage lines. Their section shall be in proportion to the drainage they will have to discharge. Outside the dam, if excavated in soil, they shall be constructed like the downstream drain, but, if in rock, they may be left as simple excavations, care being taken to divert all surface flow from them.

The main rear drain shall lead from the deepest part of the foundation where the main stream is crossed by the dam, and shall be constructed with particular care, so that its flow shall never be obstructed, A flushing pool shall be constructed at its head, in order to assist in keeping it clear.

18. General.—The longitudinal and cross sections of the various drains shall be clearly shown on the drawings, and the drains themselves shall be constructed in strict accordance with them, except when deviations are permitted in writing by the Engineer. The whole of the excavations for the drains shall be inspected by the Engineer, and they shall not be filled in until he has certified in writing that he has passed them.

III. THE SUPERSTRUCTURE OF THE DAM.

19. The Sections of the Dam.—The sections of the dam shall be clearly shown on the drawings, the work shall be constructed in strict accordance with them, and no deviation from them shall be permitted except with the written permission of the Chief Engineer. These sections shall show the limits of the hearting and the thickness of the casings, and shall clearly indicate all levels, slopes and dimensions.

20. The Material of the Dam.—The hearting shall consist of a mixture of pure clay and pure grit in the proportion of 1 to 1, but, where these materials are not procurable, the best available within half a mile of the site shall be used, in such proportions as will result in an equivalent mixture. These soils shall be free from all slushy, salty or powdery material, rubbish and vegetation.

The casings shall consist of the same materials as the hearting, but their mixture shall be equivalent to one of 1 part of pure clay to 2 parts of pure grit. The whole section of the dam shall be constructed at the same time, and patchings-on shall not be allowed except with the written permission of the Engineer.

21. Mixing and Spreading.—The clay shall be first deposited evenly to the proper thickness on the wetted surface of the last completed layer, and shall be then covered evenly with the proper thickness of grit, all clods and large particles first being broken up. The two shall be then intimately and thoroughly incorporated with each other by hand, by harrows or by inverting ploughs, as may be directed, so as to form one homogeneous mixture free from all stratification. If, after the completion of the finished layer, such stratification is detected, the layer shall be excavated and properly remade. After the mixture has been completed, the layer under construction shall be levelled and made uniform by hand or by harrows.

22. Consolidation.—On the completion of the mixture, the layer shall be thoroughly and evenly consolidated by rollers, so that no further compression by them is practicable and so that a loaded cart travelling over it shall not make a rut of perceptible depth on it. In places where the roller cannot work, the consolidation shall be effected by uniform ramming, which shall be similarly continued until further compression is not practicable by it. The rammers shall work in unison, ramming first on one side of them and then on the other, and shall advance slowly, thus kneading the earthwork together. As ramming is inferior to rolling, it shall be limited to as small an area as possible. The layers, when rammed, shall not exceed 3 inches in thickness.

In order to avoid an excessive and sudden amount of settlement, no part of the embankment shall be raised vertically more than 30 feet in one season except with the written permission of the Engineer.

During construction the layers shall be made at least 18 inches wider on each side of the dam than the designed section, and this extra width shall be dressed off after the embankment has been raised at least 5 feet higher than the layer concerned.

23. Watering.—The quantity of water used shall be rigidly limited to the amount sufficient to unite the new layer with the old one on which it is constructed, and it shall be evenly distributed over the latter just before the construction of the former is commenced.

Should the new layer during consolidation crack or move in front of the roller, this is an indication that too much water has been used, and the new layer shall be then dug up; its material shall be allowed to dry partially, and shall be then levelled and reconsolidated. Similarly, should the material be found on inspection of the weekly trial pits (clause 26) to be wet, it shall be cut out and remade, unless the Engineer by written order permits it to remain.

Should the materials of the dam be brought on to it in too dry a state to be thoroughly consolidated, they shall be watered *in situ*, and shall be utilised when in a proper condition. Soils which are naturally in too damp a condition to be used shall be excavated *in situ* and allowed to dry until they are in a fit state for the work.

After the completion of a portion of the embankment and before the dressing off of the slopes to proper section, water shall be poured down the slopes for several days until the earthwork is not able to absorb more moisture, so that the surface may be thus thoroughly compacted.

24. Thickness of Layers.—The materials of the layers shall be so deposited in thickness that, when thoroughly consolidated by ordinary

rollers, the finished layers shall have a thickness of not more than 5 inches for the top 30 feet in height of the dam, of not more than 4 inches for the next 30 feet, and of not more than 3 inches for the remainder of the base of the dam.

When steam rollers are employed, these thicknesses of the layers may be increased by fifty per cent.

25. Slopes of Layers.—The surface of the layers shall be level for a width of about one-sixteenth of the section on each side of the centre line of the dam; on the downstream side, it shall slope up at an inclination not exceeding 1 in 10; and on the upstream side, at an inclination which shall make the upstream edge level with the downstream one.

26. Testing the Construction of the Dam.—At the close of each week the work constructed during it shall be tested by means of small trial pits, 2 feet by 2 feet, which shall be excavated throughout its depth, and any change in construction or alteration of the completed work thus proved to be necessary shall be carried out in accordance with the written instructions of the Engineer.

27. Junctions of Earthwork.—Junctions shall be avoided as much as possible, but, where unavoidable, they shall be thus constructed:—

(*a*) *Cross-sectional Junctions.*—The loose surface earth of the end slope of the old embankment shall be entirely removed, and that slope shall be thereafter cut into a series of joggles and tongues sloping vertically up it. The excavated surface shall be well wetted and the new earthwork consolidated in intimate union with it.

(*b*) *Longitudinal Junctions.*—All loose surface earth shall be removed, and the solid surface of the old embankment shall be cut into a series of benchings of irregular width and depth, which shall be wetted. The new earthwork shall be then constructed in layers sloping steeply on to the old embankment, and shall be consolidated in intimate union with it.

Such junctions shall not be raised more than 20 feet in height in one season, and in the case of (*a*), they shall be broken up into vertical sections not exceeding 10 feet in height and separated by horizontal breaks of not less than 50 feet, over which the subsequent work shall lap by a distance of not less than 50 feet.

Where an old dam has to be raised, the loose top surface shall be entirely removed, and one or more key trenches, not less than 4 feet wide at bottom and 3 feet deep, and with slightly sloping sides, shall be excavated parallel to the centre line, and shall be carefully filled with the most retentive material procurable within half a mile of the site, and this shall be thoroughly rammed before the main part of the new earthwork is commenced.

28. Finishing off the Embankment.—The dam shall be constructed to the correct lines, widths and levels, and due allowance for settlement shall be made; its slopes shall be dressed uniformly to 3 inches extra to the designed width, and shall be then rammed to that width. The top shall be properly finished off, and shall be covered with half an inch of coarse sand, which shall be well rolled.

During construction arrangements shall be made for the prevention of water lodging on any part of the dam, and of the guttering of the slopes by rainfall; for filling in at once any settlements or cracks that may occur; and, generally, for maintaining the whole structure in thoroughly good order. The work shall proceed uniformly and regularly in as long continuous lengths as practicable.

The downstream slope of the dam shall be turfed, or sown, with fine grass of a binding character, and one which will thrive in this situation. Outside the F. S. contour of the reservoir, the dam shall be enclosed by a good hedge.

Proper arrangements for all works roads shall be made for use during the construction and after the completion of the dam.

29. Excavations for Materials.—Excavation shall not be permitted within a width equal to twice the height of the dam from its downstream toe, nor within one equal to four times the height of the dam from its upstream toe. At the side next the dam the depth of such excavations shall not exceed 5 feet, and shall not be increased therefrom at a greater rate than 1 in 10. The excavation pits on the downstream side shall not be continuous with each other, so that the formation of scour channels may be thus avoided.

The entire stripping off of the water-tight cover of pervious strata shall not be permitted on the reservoir side within a minimum width equal to ten times the height of the dam from its upstream toe.

All excavated pits shall be arranged in neat lines and in blocks to facilitate the record of their measurements.

IV. Pitching.

30. The Extent and Thickness of the Pitching.—The pitching shall extend from and to the levels, and shall be of the thickness shown on the drawings, and everywhere that thickness shall be represented by single through stones of the full depth. Whenever practicable, the pitching shall not be constructed until the embankment has had a full year in which to settle.

31. The Stone.—The pitching shall consist of sound, hard and durable stone which will not weather and will not deteriorate when exposed to

the action of water. It shall be roughly hammer-dressed, so that all large projections shall be removed in order that the stones may meet all round their bases for a depth at least one-quarter of their height and completely cover and protect the embankment. The stones shall be as regular as possible in horizontal and vertical section, and at least nine-tenths of them shall have a horizontal section at the base of not less than 50 square inches for 12-inch pitching; 65 square inches for 15-inch pitching; 80 square inches for 18-inch pitching; and 100 square inches for 2-feet pitching.

32. Laying.—The stones shall be firmly bedded on a layer of sound, hard murum or quarry spauls at least 6 inches in thickness, and shall be laid with their broadest ends downwards, so that they may meet all round their bases for a minimum depth equal to one-quarter of their height. They shall be hammer-dressed so as to fit closely to each other at their bases, and shall be malleted securely against each other and on to their seat, and so that their bases shall be parallel to the slope of the dam, and that, when struck by a heavy hammer, they shall not be disturbed. The stones shall be laid with the longest dimension of their bases as nearly parallel to the axis of the dam as possible, and shall break joint in every direction, so that long, unbroken joints shall be avoided. After the laying of the pitching has been inspected and passed, any large spaces between the tops of the stones shall be filled each with a single large chip well driven home, so that the stones may be firmly wedged to each other, and so that the upper surface of the pitching shall present a fairly regular appearance free from large projections. The pitching, thus completed, shall be finally inspected and passed.

The face slope of the completed pitching shall be that designed for the dam, and the various thicknesses of the bedding and the stones shall be allowed for accordingly in the earthwork of the embankment.

33. Foundation and Top Courses.—The foundation course shall consist of a line of headers, the depth of which shall not be less than 1 foot in excess of that of the pitching stones abutting on them. They shall be roughly squared and shall be fixed on a bed of spauls in a small trench cut to take their extra depth. Where the ground is soft and liable to erosion by wave action, an apron of large quarry spauls, or medium-sized rubble, shall be formed over and for at least 3 feet beyond the foundation course so as to protect it.

The top course shall consist of a line of headers, the depth of which shall not be less than 9 inches in excess of that of the pitching at the top, and shall project by that amount beyond the face of the pitching. They shall be roughly squared, and shall be fitted together with close joints in one continuous line parallel to the top of the dam.

APPENDIX 19.

(*Vide* Chapter II., paragraph 92.)

TABLES OF THE CROSS-SECTIONAL AREAS OF PUDDLE TRENCHES.

Formula A = (B + SD)D.

Where A = the cross-sectional area in square feet;
B = the bottom-width in feet;
S = the ratio of the side-slopes to unity;
D = the depth in feet.

TABLE I.

Bottom-width = 10 feet; Side-slopes = ¼ to 1.
A = (10 + 0·25 D)D.

SQUARE FEET.

Depth in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	1·002	2·010	3·022	4·040	5·062	6·090	7·122	8·160	9·202
1	10·250	11·302	12·360	13·422	14·490	15·562	16·640	17·722	18·810	19·902
2	21·000	22·107	23·210	24·322	25·440	26·562	27·690	28·822	29·960	31·102
3	32·250	33·402	34·560	35·722	36·890	38·062	39·240	40·422	41·610	42·802
4	44·000	45·202	46·410	47·622	48·840	50·062	51·290	52·522	53·760	55·002
5	56·250	57·502	58·760	60·022	61·290	62·562	63·840	65·122	66·410	67·702
6	69·000	70·302	71·610	72·922	74·240	75·562	76·890	78·222	79·560	80·902
7	82·250	83·602	84·960	86·322	87·690	89·062	90·440	91·822	93·210	94·602
8	96·000	97·402	98·810	100·222	101·640	103·062	104·490	105·922	107·360	108·802
9	110·250	111·702	113·160	114·622	116·090	117·562	119·040	120·522	122·010	123·502
10	125·000	126·502	128·010	129·522	131·040	132·562	134·090	135·622	137·160	138·702
11	140·250	141·802	143·360	144·922	146·490	148·062	149·640	151·222	152·810	154·402
12	156·000	157·602	159·210	160·822	162·440	164·062	165·690	167·322	168·960	170·602
13	172·250	173·902	175·560	177·222	178·890	180·562	182·240	183·922	185·610	187·302
14	189·000	190·702	192·410	194·122	195·840	197·562	199·290	201·022	202·760	204·502
15	206·250	208·002	209·760	211·522	213·290	215·062	216·840	218·622	220·410	222·202
16	224·000	225·802	227·610	229·422	231·240	233·062	234·890	236·722	238·560	240·402
17	242·250	244·102	245·960	247·822	249·690	251·562	253·440	255·322	257·210	259·102
18	261·000	262·902	264·810	266·722	268·640	270·562	272·490	274·422	276·360	278·302
19	280·250	282·202	284·160	286·122	288·090	290·062	292·040	294·022	296·010	298·002
20	300·000	302·002	304·010	306·022	308·040	310·062	312·090	314·122	316·160	318·202
21	320·250	322·302	324·360	326·422	328·490	330·562	332·640	334·722	336·810	338·902
22	341·000	343·102	345·210	347·322	349·440	351·562	353·690	355·822	357·960	360·102
23	362·250	364·402	366·560	368·722	370·890	373·062	375·240	377·422	379·610	381·802
24	384·000	386·202	388·410	390·622	392·840	395·062	397·290	399·522	401·760	404·002
25	406·250	408·502	410·760	413·022	415·290	417·562	419·840	422·122	424·410	426·702
26	429·000	431·302	433·610	435·922	438·240	440·562	442·890	445·222	447·560	449·902
27	452·250	454·602	456·960	459·322	461·690	464·062	466·440	468·822	471·210	473·602
28	476·000	478·402	480·810	483·222	485·640	488·062	490·490	492·922	495·360	497·802
29	500·250	502·702	505·160	507·622	510·090	512·562	515·040	517·522	520·010	522·502
30	525·000	527·502	530·010	532·522	535·040	537·562	540·090	542·622	545·160	547·702
31	550·250	552·802	555·360	557·922	560·490	563·062	565·640	568·222	570·810	573·402
32	576·000	578·602	581·210	583·822	586·440	589·062	591·690	594·322	596·960	599·602
33	602·250	604·902	607·560	610·222	612·890	615·562	618·240	620·922	623·610	626·302
34	629·000	631·702	634·410	637·122	639·840	642·562	645·290	648·022	650·760	653·502
35	656·250	659·002	661·760	664·522	667·290	670·062	672·840	675·622	678·410	681·202
36	684·000	686·802	689·610	692·422	695·240	698·062	700·890	703·722	706·560	709·402
37	712·250	715·102	717·960	720·822	723·690	726·562	729·440	732·322	735·210	738·102
38	741·000	743·902	746·810	749·722	752·640	755·562	758·490	761·422	764·360	767·302
39	770·250	773·202	776·160	779·122	782·090	785·062	788·040	791·022	794·010	797·002
40	800·000	803·002	806·010	809·022	812·040	815·062	818·090	821·122	824·160	827·202

NOTE.—For bottom-widths differing from 10 feet, multiply the difference in bottom-width by the depth, and add or subtract the product, as the case may be, to or from the tabular quantities.

TABLE II.

Bottom-width = 10 feet; Side-slopes = ½ to 1.

A = (10 + 0·5 D) D sq. ft.

SQUARE FEET.

Depth in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	1·005	2·020	3·045	4·080	5·125	6·180	7·245	8·320	9·405
1	10·500	11·605	12·720	13·845	14·980	16·125	17·280	18·445	19·620	20·805
2	22·000	23·205	24·420	25·645	26·880	28·125	29·380	30·645	31·920	33·205
3	34·500	35·805	37·120	38·445	39·780	41·125	42·480	43·845	45·220	46·605
4	48·000	49·405	50·820	52·245	53·680	55·125	56·580	58·045	59·520	61·005
5	62·500	64·005	65·520	67·045	68·580	70·125	71·680	73·245	74·820	76·405
6	78·000	79·605	81·220	82·845	84·480	86·125	87·780	89·445	91·120	92·805
7	94·500	96·205	97·920	99·645	101·380	103·125	104·880	106·645	108·420	110·205
8	112·000	113·805	115·620	117·445	119·280	121·125	122·980	124·845	126·720	128·605
9	130·500	132·405	134·320	136·245	138·180	140·125	142·080	144·045	146·020	148·005
10	150·000	152·005	154·020	156·045	158·080	160·125	162·180	164·245	166·320	168·405
11	170·500	172·605	174·720	176·845	178·980	181·125	183·280	185·445	187·620	189·805
12	192·000	194·205	196·420	198·645	200·880	203·125	205·380	207·645	209·920	212·205
13	214·500	216·805	219·120	221·445	223·780	226·125	228·480	230·845	233·220	235·605
14	238·000	240·405	242·820	245·245	247·680	250·125	252·580	255·045	257·520	260·005
15	262·500	265·005	267·520	270·045	272·580	275·125	277·680	280·245	282·820	285·405
16	288·000	290·605	293·220	295·845	298·480	301·125	303·780	306·445	309·120	311·805
17	314·500	317·205	319·920	322·645	325·380	328·125	330·880	333·645	336·420	339·205
18	342·000	344·805	347·620	350·445	353·280	356·125	358·980	361·845	364·720	367·605
19	370·500	373·405	376·320	379·245	382·180	385·125	388·080	391·045	394·020	397·005
20	400·000	403·005	406·020	409·045	412·080	415·125	418·180	421·245	424·320	427·405
21	430·500	433·605	436·720	439·845	442·980	446·125	449·280	452·445	455·620	458·805
22	462·000	465·205	468·420	471·645	474·880	478·125	481·380	484·645	487·920	491·205
23	494·500	497·805	501·120	504·445	507·780	511·125	514·480	517·845	521·220	524·605
24	528·000	531·405	534·820	538·245	541·680	545·125	548·580	552·045	555·520	559·005
25	562·500	566·005	569·520	573·045	576·580	580·125	583·680	587·245	590·820	594·405
26	598·000	601·605	605·220	608·845	612·480	616·125	619·780	623·445	627·120	630·805
27	634·500	638·205	641·920	645·645	649·380	653·125	656·880	660·645	664·420	668·205
28	672·000	675·805	679·620	683·445	687·280	691·125	694·980	698·845	702·720	706·605
29	710·500	714·405	718·320	722·245	726·180	730·125	734·080	738·045	742·020	746·005
30	750·000	754·005	758·020	762·045	766·080	770·125	774·180	778·245	782·320	786·405
31	790·500	794·605	798·720	802·845	806·980	811·125	815·280	819·445	823·620	827·805
32	832·000	836·205	840·420	844·645	848·880	853·125	857·380	861·645	865·920	870·205
33	874·500	878·805	883·120	887·445	891·780	896·125	900·480	904·845	909·220	913·605
34	918·000	922·405	926·820	931·245	935·680	940·125	944·580	949·045	953·520	958·005
35	962·500	967·005	971·520	976·045	980·580	985·125	989·680	994·245	998·820	1,003·405
36	1,008·000	1,012·605	1,017·220	1,021·845	1,026·480	1,031·125	1,035·780	1,040·445	1,045·120	1,049·805
37	1,054·500	1,059·205	1,063·920	1,068·645	1,073·380	1,078·125	1,082·880	1,087·645	1,092·420	1,097·205
38	1,102·000	1,106·805	1,111·620	1,116·445	1,121·280	1,126·125	1,130·980	1,135·845	1,140·720	1,145·605
39	1,150·500	1,155·405	1,160·320	1,165·245	1,170·180	1,175·125	1,180·080	1,185·045	1,190·020	1,195·005
40	1,200·000	1,205·005	1,210·020	1,215·045	1,220·080	1,225·125	1,230·180	1,235·245	1,240·320	1,245·405

NOTE.—For bottom-widths differing from 10 feet, multiply the difference in bottom-width by the depth, and add or subtract the product, as the case may be, to or from the tabular quantities

APPENDIX 20.

(*Vide* Chapter II., paragraph 73.)

TABLES OF THE CROSS-SECTIONAL AREAS OF DAM EMBANKMENTS.

$$\text{Formula:—} A = \left\{ T + H\left(\frac{S_1 + S_2}{2}\right) \right\} H$$

Where A = the area of the section in square feet;

T = the top-width of the dam in feet;

S_1, S_2 = the ratios of the side-slopes of the dam;

H = the height of the dam in feet, measured from the cleared foundation to the top of the dam and including the allowance for settlement.

TABLE I.

Top-width, 6 feet; Upstream slope, 2 to 1; Downstream slope, 1½ to 1.

$A = (6 + 1{\cdot}75\ H)\ H$ sq. ft.

SQUARE FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·617	1·270	1·957	2·680	3·437	4·230	5·057	5·920	6·817
1	7·750	8·717	9·720	10·757	11·830	12·937	14·080	15·257	16·470	17·717
2	19·000	20·317	21·670	23·057	24·480	25·937	27·430	28·957	30·520	32·117
3	33·750	35·417	37·120	38·857	40·630	42·437	44·280	46·157	48·070	50·017
4	52·000	54·017	56·070	58·157	60·280	62·437	64·630	66·857	69·120	71·417
5	73·750	76·117	78·520	80·957	83·430	85·937	88·480	91·057	93·670	96·317
6	99·000	101·717	104·470	107·257	110·080	112·937	115·830	118·757	121·720	124·717
7	127·750	130·817	133·920	137·057	140·230	143·437	146·680	149·957	153·270	156·617
8	160·000	163·417	166·870	170·357	173·880	177·437	181·030	184·657	188·320	192·017
9	195·750	199·517	203·320	207·157	211·030	214·937	218·880	222·857	226·870	230·917
10	235·000	239·117	243·270	247·457	251·680	255·937	260·230	264·557	268·920	273·317
11	277·750	282·217	286·720	291·257	295·830	300·437	305·080	309·757	314·470	319·217
12	324·000	328·817	333·670	338·557	343·480	348·437	353·430	358·457	363·520	368·617
13	373·750	378·917	384·120	389·357	394·630	399·937	405·280	410·657	416·070	421·517
14	427·000	432·517	438·070	443·657	449·280	454·937	460·630	466·357	472·120	477·917
15	483·750	489·617	495·520	501·457	507·430	513·437	519·480	525·557	531·670	537·817

Note.—For top widths differing from 6 feet, multiply the difference in top width by the height, and add or subtract the product, as the case may be, to or from the tabular quantities.

TABLE II

Top-width, 6 feet; Upstream slope, 2½ to 1; Downstream slope, 2 to 1.
$A = (6 + 2{\cdot}25\ H)\ H$ sq. ft.

SQUARE FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·622	1·290	2·002	2·760	3·562	4·410	5·302	6·240	7·222
1	8·250	9·322	10·440	11·602	12·810	14·062	15·360	16·702	18·090	19·522
2	21·000	22·522	24·090	25·702	27·360	29·062	30·810	32·602	34·440	36·322
3	38·250	40·222	42·240	44·302	46·410	48·562	50·760	53·002	55·290	57·622
4	60·000	62·422	64·890	67·402	69·960	72·562	75·210	77·902	80·640	83·422
5	86·250	89·122	92·040	95·002	98·010	101·062	104·160	107·302	110·490	113·722
6	117·000	120·322	123·690	127·102	130·560	134·062	137·610	141·202	144·840	148·522
7	152·250	156·022	159·840	163·702	167·610	171·562	175·560	179·602	183·690	187·822
8	192·000	196·222	200·490	204·802	209·160	213·562	218·010	222·502	227·040	231·622
9	236·250	240·922	245·640	250·402	255·210	260·062	264·960	269·902	274·890	279·922
10	285·000	290·122	295·290	300·502	305·760	311·062	316·410	321·802	327·240	332·722
11	338·250	343·822	349·440	355·102	360·810	366·562	372·360	378·202	384·090	390·022
12	396·000	402·022	408·090	414·202	420·360	426·562	432·810	439·102	445·440	451·822
13	458·250	464·722	471·240	477·802	484·410	491·062	497·760	504·502	511·290	518·122
14	525·000	531·922	538·890	545·902	552·960	560·062	567·210	574·402	581·640	588·922
15	596·250	603·622	611·040	618·502	626·010	633·562	641·160	648·802	656·490	664·222
16	672·000	679·822	687·690	695·602	703·560	711·562	719·610	727·702	735·840	744·022
17	752·250	760·522	768·840	777·202	785·610	794·062	802·560	811·102	819·690	828·322
18	837·000	845·722	854·490	863·302	872·160	881·062	890·010	899·002	908·040	917·122
19	926·250	935·422	944·640	953·902	963·210	972·562	981·960	991·402	1,000·890	1,010·422
20	1,020·000	1,029·622	1,039·290	1,049·002	1,058·760	1,068·562	1,078·410	1,088·302	1,098·240	1,108·222
21	1,118·250	1,128·322	1,138·440	1,148·602	1,158·810	1,169·062	1,179·360	1,189·702	1,200·090	1,210·522
22	1,221·000	1,231·522	1,242·090	1,252·702	1,263·360	1,274·062	1,284·810	1,295·602	1,306·440	1,317·322
23	1,328·250	1,339·222	1,350·240	1,361·302	1,372·410	1,383·562	1,394·760	1,406·002	1,417·290	1,428·622
24	1,440·000	1,451·422	1,462·890	1,474·402	1,485·960	1,497·562	1,509·210	1,520·902	1,532·640	1,544·422
25	1,556·250	1,568·122	1,580·040	1,592·002	1,604·010	1,616·062	1,628·160	1,640·302	1,652·490	1,664·722

NOTE.—For top widths differing from 6 feet, multiply the difference in top width by the height, and add or subtract the product, as the case may be, to or from the tabular quantities.

TABLE III.

Top-width, 8 feet; Upstream slope, 3 to 1; Downstream slope, 2 to 1.
$A = (8 + 2{\cdot}5\ H)\ H$ sq. ft.

SQUARE FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·825	1·700	2·625	3·600	4·625	5·700	6·825	8·000	9·225
1	10·500	11·825	13·200	14·625	16·100	17·625	19·200	20·825	22·500	24·225
2	26·000	27·825	29·700	31·625	33·600	35·625	37·700	39·825	42·000	44·225
3	46·500	48·825	51·200	53·625	56·100	58·625	61·200	63·825	66·500	69·225
4	72·000	74·825	77·700	80·625	83·600	86·625	89·700	92·825	96·000	99·225
5	102·500	105·825	109·200	112·625	116·100	119·625	123·200	126·825	130·500	134·225
6	138·000	141·825	145·700	149·625	153·600	157·625	161·700	165·825	170·000	174·225
7	178·500	182·825	187·200	191·625	196·100	200·625	205·200	209·825	214·500	219·225
8	224·000	228·825	233·700	238·625	243·600	248·625	253·700	258·825	264·000	269·225
9	274·500	279·825	285·200	290·625	296·100	301·625	307·200	312·825	318·500	324·225
10	330·000	335·825	341·700	347·625	353·600	359·625	365·700	371·825	378·000	384·225
11	390·500	396·825	403·200	409·625	416·100	422·625	429·200	435·825	442·500	449·225
12	456·000	462·825	469·700	476·625	483·600	490·625	497·700	504·825	512·000	519·225
13	526·500	533·825	541·200	548·625	556·100	563·625	571·200	578·825	586·500	594·225
14	602·000	609·825	617·700	625·625	633·600	641·625	649·700	657·825	666·000	674·225
15	682·500	690·825	699·200	707·625	716·100	724·625	733·200	741·825	750·500	759·225
16	768·000	776·825	785·700	794·625	803·600	812·625	821·700	830·825	840·000	849·225
17	858·500	867·825	877·200	886·625	896·100	905·625	915·200	924·825	934·500	944·225
18	954·000	963·825	973·700	983·625	993·600	1,003·625	1,013·700	1,023·825	1,034·000	1,044·225
19	1,054·500	1,064·825	1,075·200	1,085·625	1,096·100	1,106·625	1,117·200	1,127·825	1,138·500	1,149·225
20	1,160·000	1,170·825	1,181·700	1,192·625	1,203·600	1,214·625	1,225·700	1,236·825	1,248·000	1,259·225
21	1,270·500	1,281·825	1,293·200	1,304·625	1,316·100	1,327·625	1,339·200	1,350·825	1,362·500	1,374·225
22	1,386·000	1,397·825	1,409·700	1,421·625	1,433·600	1,445·625	1,457·700	1,469·825	1,482·000	1,494·225
23	1,506·500	1,518·825	1,531·200	1,543·625	1,556·100	1,568·625	1,581·200	1,593·825	1,606·500	1,619·225
24	1,632·000	1,644·825	1,657·700	1,670·625	1,683·600	1,696·625	1,709·700	1,722·825	1,736·000	1,749·225
25	1,762·500	1,775·825	1,789·200	1,802·625	1,816·100	1,829·625	1,843·200	1,856·825	1,870·500	1,884·225
26	1,898·000	1,911·825	1,925·700	1,939·625	1,953·600	1,967·625	1,981·700	1,995·825	2,010·000	2,024·225
27	2,038·500	2,052·825	2,067·200	2,081·625	2,096·100	2,110·625	2,125·200	2,139·825	2,154·500	2,169·225
28	2,184·000	2,198·825	2,213·700	2,228·625	2,243·600	2,258·625	2,273·700	2,288·825	2,304·000	2,319·225
29	2,334·500	2,349·825	2,365·200	2,380·625	2,396·100	2,411·625	2,427·200	2,442·825	2,458·500	2,474·225
30	2,490·000	2,505·825	2,521·700	2,537·625	2,553·600	2,569·625	2,585·700	2,601·825	2,618·000	2,634·225
31	2,650·500	2,666·825	2,683·200	2,699·625	2,716·100	2,732·625	2,749·200	2,765·825	2,782·500	2,799·225
32	2,816·000	2,832·825	2,849·700	2,866·625	2,883·600	2,900·625	2,917·700	2,934·825	2,952·000	2,969·225
33	2,986·500	3,003·825	3,021·200	3,038·625	3,056·100	3,073·625	3,091·200	3,108·825	3,126·500	3,144·225
34	3,162·000	3,179·825	3,197·700	3,215·625	3,233·600	3,251·625	3,269·700	3,287·825	3,306·000	3,324·225
35	3,342·500	3,360·825	3,379·200	3,397·625	3,416·100	3,434·625	3,453·200	3,471·825	3,490·500	3,509·225
36	3,528·000	3,546·825	3,565·700	3,584·625	3,603·600	3,622·625	3,641·700	3,660·825	3,680·000	3,699·225
37	3,718·500	3,737·825	3,757·200	3,776·625	3,796·100	3,815·625	3,835·200	3,854·825	3,874·500	3,894·225
38	3,914·000	3,933·825	3,953·700	3,973·625	3,993·600	4,013·625	4,033·700	4,053·825	4,074·000	4,094·225
39	4,114·500	4,134·825	4,155·200	4,175·625	4,196·100	4,216·625	4,237·200	4,257·825	4,278·500	4,299·225
40	4,320·000	4,340·825	4,361·700	4,382·625	4,403·600	4,424·625	4,445·700	4,466·825	4,488·000	4,509·225
41	4,530·500	4,551·825	4,573·200	4,594·625	4,616·100	4,637·625	4,659·200	4,680·825	4,702·500	4,724·225
42	4,746·000	4,767·825	4,789·700	4,811·625	4,833·600	4,855·625	4,877·700	4,899·825	4,922·000	4,944·225
43	4,966·500	4,988·825	5,011·200	5,033·625	5,056·100	5,078·625	5,101·200	5,123·825	5,146·500	5,169·225
44	5,192·000	5,214·825	5,237·700	5,260·625	5,283·600	5,306·625	5,329·700	5,352·825	5,376·000	5,399·225
45	5,422·500	5,445·825	5,469·200	5,492·625	5,516·100	5,539·625	5,563·200	5,586·825	5,610·500	5,634·225
46	5,658·000	5,681·825	5,705·700	5,729·625	5,753·600	5,777·625	5,801·700	5,825·825	5,850·000	5,874·225
47	5,898·500	5,922·825	5,947·200	5,971·625	5,996·100	6,020·625	6,045·200	6,069·825	6,094·500	6,119·225
48	6,144·000	6,168·825	6,193·700	6,218·625	6,243·600	6,268·625	6,293·700	6,318·825	6,344·000	6,369·225
49	6,394·500	6,419·825	6,445·200	6,470·625	6,496·100	6,521·625	6,547·200	6,572·825	6,598·500	6,624·225
50	6,650·000	6,675·825	6,701·700	6,727·625	6,753·600	6,779·625	6,805·700	6,831·825	6,858·000	6,884·225

NOTE.—For the widths differing from 8 feet, multiply the difference in top width by the height, and add or subtract the product, as the case may be, to or from the tabular quantities.

TABLE IV.

Top-width, 10 feet; Upstream slope, 3 to 1; Downstream slope, 2 to 1.
A = (10 + 2·5 H) H sq. ft.

SQUARE FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	1·025	2·100	3·225	4·400	5·625	6·900	8·225	9·600	11·025
1	12·500	14·025	15·600	17·225	18·900	20·625	22·400	24·225	26·100	28·025
2	30·000	32·025	34·100	36·225	38·400	40·625	42·900	45·225	47·600	50·025
3	52·500	55·025	57·600	60·225	62·900	65·625	68·400	71·225	74·100	77·025
4	80·000	83·025	86·100	89·225	92·400	95·625	98·900	102·225	105·600	109·025
5	112·500	116·025	119·600	123·225	126·900	130·625	134·400	138·225	142·100	146·025
6	150·000	154·025	158·100	162·225	166·400	170·625	174·900	179·225	183·600	188·025
7	192·500	197·025	201·600	206·225	210·900	215·625	220·400	225·225	230·100	235·025
8	240·000	245·025	250·100	255·225	260·400	265·625	270·900	276·225	281·600	287·025
9	292·500	298·025	303·600	309·225	314·900	320·625	326·400	332·225	338·100	344·025
10	350·000	356·025	362·100	368·225	374·400	380·625	386·900	393·225	399·600	406·025
11	412·500	419·025	425·600	432·225	438·900	445·625	452·400	459·225	466·100	473·025
12	480·000	487·025	494·100	501·225	508·400	515·625	522·900	530·225	537·600	545·025
13	552·500	560·025	567·600	575·225	582·900	590·625	598·400	606·225	614·100	622·025
14	630·000	638·025	646·100	654·225	662·400	670·625	678·900	687·225	695·600	704·025
15	712·500	721·025	729·600	738·225	746·900	755·625	764·400	773·225	782·100	791·025
16	800·000	809·025	818·100	827·225	836·400	845·625	854·900	864·225	873·600	883·025
17	892·500	902·025	911·600	921·225	930·900	940·625	950·400	960·225	970·100	980·025
18	990·000	1,000·025	1,010·100	1,020·225	1,030·400	1,040·625	1,050·900	1,061·225	1,071·600	1,082·025
19	1,092·500	1,103·025	1,113·600	1,124·225	1,134·900	1,145·625	1,156·400	1,167·225	1,178·100	1,189·025
20	1,200·000	1,211·025	1,222·100	1,233·225	1,244·400	1,255·625	1,266·900	1,278·225	1,289·600	1,301·025
21	1,312·500	1,324·025	1,335·600	1,347·225	1,358·900	1,370·625	1,382·400	1,394·225	1,406·100	1,418·025
22	1,430·000	1,442·025	1,454·100	1,466·225	1,478·400	1,490·625	1,502·900	1,515·225	1,527·600	1,540·025
23	1,552·500	1,565·025	1,577·600	1,590·225	1,602·900	1,615·625	1,628·400	1,641·225	1,654·100	1,667·025
24	1,680·000	1,693·025	1,706·100	1,719·225	1,732·400	1,745·625	1,758·900	1,772·225	1,785·600	1,799·025
25	1,812·500	1,826·025	1,839·600	1,853·225	1,866·900	1,880·625	1,894·400	1,908·225	1,922·100	1,936·025
26	1,950·000	1,964·025	1,978·100	1,992·225	2,006·400	2,020·625	2,034·900	2,049·225	2,063·600	2,078·025
27	2,092·500	2,107·025	2,121·600	2,136·225	2,150·900	2,165·625	2,180·400	2,195·225	2,210·100	2,225·025
28	2,240·000	2,255·025	2,270·100	2,285·225	2,300·400	2,315·625	2,330·900	2,346·225	2,361·600	2,377·025
29	2,392·500	2,408·025	2,423·600	2,439·225	2,454·900	2,470·625	2,486·400	2,502·225	2,518·100	2,534·025
30	2,550·000	2,566·025	2,582·100	2,598·225	2,614·400	2,630·625	2,646·900	2,663·225	2,679·600	2,696·025
31	2,712·500	2,729·025	2,745·600	2,762·225	2,778·900	2,795·625	2,812·400	2,829·225	2,846·100	2,863·025
32	2,880·000	2,897·025	2,914·100	2,931·225	2,948·400	2,965·625	2,982·900	3,000·225	3,017·600	3,035·025
33	3,052·500	3,070·025	3,087·600	3,105·225	3,122·900	3,140·625	3,158·400	3,176·225	3,194·100	3,212·025
34	3,230·000	3,248·025	3,266·100	3,284·225	3,302·400	3,320·625	3,338·900	3,357·225	3,375·600	3,394·025
35	3,412·500	3,431·025	3,449·600	3,468·225	3,486·900	3,505·625	3,524·400	3,543·225	3,562·100	3,581·025
36	3,600·000	3,619·025	3,638·100	3,657·225	3,676·400	3,695·625	3,714·900	3,734·225	3,753·600	3,773·025
37	3,792·500	3,812·025	3,831·600	3,851·225	3,870·900	3,890·625	3,910·400	3,930·225	3,950·100	3,970·025
38	3,990·000	4,010·025	4,030·100	4,050·225	4,070·400	4,090·625	4,110·900	4,131·225	4,151·600	4,172·025
39	4,192·500	4,213·025	4,233·600	4,254·225	4,274·900	4,295·625	4,316·400	4,337·225	4,358·100	4,379·025
40	4,400·000	4,421·025	4,442·100	4,463·225	4,484·400	4,505·625	4,526·900	4,548·225	4,569·600	4,591·025
41	4,612·500	4,634·025	4,656·600	4,677·225	4,698·900	4,720·625	4,742·400	4,764·225	4,786·100	4,808·025
42	4,830·000	4,852·025	4,874·100	4,896·225	4,918·400	4,940·625	4,962·900	4,985·225	5,007·600	5,030·025
43	5,052·500	5,075·025	5,097·600	5,120·225	5,142·900	5,165·625	5,188·400	5,211·225	5,234·100	5,257·025
44	5,280·000	5,303·025	5,326·100	5,349·225	5,372·400	5,395·625	5,418·900	5,442·225	5,465·600	5,489·025
45	5,512·500	5,536·025	5,559·600	5,583·225	5,606·900	5,630·625	5,654·400	5,678·225	5,702·100	5,726·025
46	5,750·000	5,774·025	5,798·100	5,822·225	5,846·400	5,870·625	5,894·900	5,919·225	5,943·600	5,968·025
47	5,992·500	6,017·025	6,041·600	6,066·225	6,090·900	6,115·625	6,140·400	6,165·225	6,190·100	6,215·025
48	6,240·000	6,265·025	6,290·100	6,315·225	6,340·400	6,365·625	6,390·900	6,416·225	6,441·600	6,467·025
49	6,492·500	6,518·025	6,543·600	6,569·225	6,594·900	6,620·625	6,646·400	6,672·225	6,698·100	6,724·025
50	6,750·000	6,776·025	6,802·100	6,828·225	6,854·400	6,880·625	6,906·900	6,933·225	6,959·600	6,986·025
51	7,012·500	7,039·025	7,065·600	7,092·225	7,118·900	7,145·625	7,172·400	7,199·225	7,226·100	7,253·025
52	7,280·000	7,307·025	7,334·100	7,361·225	7,388·400	7,415·625	7,442·900	7,470·225	7,497·600	7,525·025
53	7,552·500	7,580·025	7,607·600	7,635·225	7,662·900	7,690·625	7,718·400	7,746·225	7,774·100	7,802·025
5	7,830·000	7,858·025	7,886·100	7,914·225	7,942·400	7,970·625	7,998·900	8,027·225	8,055·600	8,084·025

TABLE IV.—*continued.*

SQUARE FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
55	3,112·500	3,141·025	3,169·600	3,198·225	3,226·900	3,255·625	3,284·400	3,313·225	3,342·100	3,371·025
56	8,400·000	8,429·025	8,458·100	8,487·225	8,516·400	8,545·625	8,574·900	8,604·225	8,633·600	8,663·025
57	3,692·500	3,722·025	3,751·600	3,781·225	3,810·900	3,840·625	3,870·400	3,900·225	3,930·100	3,960·025
58	8,990·000	9,020·025	9,050·100	9,080·225	9,110·400	9,140·625	9,170·900	9,201·225	9,231·600	9,262·025
59	9,292·500	9,323·025	9,353·600	9,384·225	9,414·900	9,445·625	9,476·400	9,507·225	9,538·100	9,569·025
60	9,600·000	9,631·025	9,662·100	9,693·225	9,724·400	9,755·625	9,786·900	9,818·225	9,849·600	9,881·025
61	9,912·500	9,944·025	9,975·600	10,007·225	10,038·900	10,070·625	10,102·400	10,134·225	10,166·100	10,198·025
62	10,230·000	10,262·025	10,294·100	10,326·225	10,358·400	10,390·625	10,422·900	10,455·225	10,487·600	10,520·025
63	10,552·500	10,585·025	10,617·600	10,650·225	10,682·900	10,715·625	10,748·400	10,781·225	10,814·100	10,847·025
64	10,880·000	10,913·025	10,946·100	10,979·225	11,012·400	11,045·625	11,078·900	11,112·225	11,145·600	11,179·025
65	11,212·500	11,246·025	11,279·600	11,313·225	11,346·900	11,380·625	11,414·400	11,448·225	11,482·100	11,516·025
66	11,550·000	11,584·025	11,618·100	11,652·225	11,686·400	11,720·625	11,754·900	11,789·225	11,823·600	11,858·025
67	11,892·500	11,927·025	11,961·600	11,996·225	12,030·900	12,065·625	12,100·400	12,135·225	12,170·100	12,205·025
68	12,240·000	12,275·025	12,310·100	12,345·225	12,380·400	12,415·625	12,450·900	12,486·225	12,521·600	12,557·025
69	12,602·500	12,623·025	12,663·600	12,699·225	12,734·900	12,770·625	12,806·400	12,842·225	12,878·100	12,914·025
70	12,950·000	12,986·025	13,022·100	13,058·225	13,094·400	13,130·625	13,166·900	13,203·225	13,239·600	13,276·025
71	13,312·500	13,349·025	13,385·600	13,422·225	13,458·900	13,495·625	13,532·400	13,569·225	13,606·100	13,643·025
72	13,680·000	13,717·025	13,754·100	13,791·225	13,828·400	13,865·625	13,902·900	13,940·225	13,977·600	14,015·025
73	14,052·500	14,090·025	14,127·600	14,165·225	14,202·900	14,240·625	14,278·400	14,316·225	14,354·100	14,392·025
74	14,430·000	14,468·025	14,506·100	14,544·225	14,582·400	14,620·625	14,658·900	14,697·225	14,735·600	14,774·025
75	14,812·500	14,851·025	14,889·600	14,928·225	14,966·900	15,005·625	15,044·400	15,083·225	15,122·100	15,161·025
76	15,200·000	15,239·025	15,278·100	15,317·225	15,356·400	15,395·625	15,434·900	15,474·225	15,513·600	15,553·025
77	15,592·500	15,632·025	15,671·600	15,711·225	15,750·900	15,790·625	15,830·400	15,870·225	15,910·100	15,950·025
78	15,990·000	16,030·025	16,070·100	16,110·225	16,150·400	16,190·625	16,230·900	16,271·225	16,311·600	16,352·025
79	16,392·500	16,433·025	16,473·600	16,514·225	16,554·900	16,595·625	16,636·400	16,677·225	16,718·100	16,759·025
80	16,800·000	16,841·025	16,882·100	16,923·225	16,964·400	17,005·625	17,046·900	17,088·225	17,129·600	17,171·025

NOTE.—For top-widths differing from 10 feet, multiply the difference in top width by the height, and add or subtract the product, as the case may be, to or from the tabular quantities.

FORM OF MEASUREMENT SHEET FOR THE ESTIMATE OF THE EMBANKMENT OF A DAM.

1	2	3	4	5	6	7	8	
Point on Section.	Height of Dam as per Section.	Depth allowed for Clearance.	Total (2) + (3).	Allowance for Settlement, $\frac{1}{20}$ × Col. 4.	Grand total height (4) + (5).	Cross-sectional Area corresponding to (6).	Length between Points on Section.	Quantity between Points on Section, (7) × (8).

APPENDIX 21.

TABLES OF THE CROSS-SECTIONAL LENGTHS OF PITCHING.

(*Vide* Chapter II., paragraph 152.)

Formula :—L = S H.

Where L is the cross-sectional length in feet; and
S, the ratio of the length of the slope to
H, the vertical height.

TABLE I.

Upstream slope = 1½ to 1; S = 1·803.

FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·180	0·361	0·541	0·721	0·901	1·082	1·262	1·442	1·623
1	1·803	1·983	2·164	2·344	2·524	2·704	2·885	3·065	3·245	3·426
2	3·606	3·786	3·967	4·147	4·327	4·507	4·688	4·868	5·048	5·229
3	5·409	5·589	5·770	5·950	6·130	6·310	6·491	6·671	6·851	7·032
4	7·212	7·392	7·573	7·753	7·933	8·113	8·294	8·474	8·654	8·835
5	9·015	9·196	9·376	9·556	9·736	9·916	10·097	10·277	10·457	10·638
6	10·818	10·998	11·179	11·359	11·539	11·719	11·900	12·080	12·260	12·441
7	12·621	12·801	12·982	13·162	13·342	13·522	13·703	13·883	14·063	14·244
8	14·424	14·604	14·785	14·965	15·145	15·325	15·506	15·686	15·866	16·047
9	16·227	16·407	16·588	16·768	16·948	17·128	17·309	17·489	17·669	17·850
10	18·030	18·210	18·391	18·571	18·751	18·931	19·112	19·292	19·472	19·653

TABLE II.

Upstream slope = 2 to 1; S = 2·236.

FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·224	0·447	0·671	0·894	1·118	1·342	1·565	1·789	2·012
1	2·235	2·460	2·683	2·907	3·130	3·354	3·578	3·801	4·025	4·248
2	4·472	4·696	4·919	5·143	5·366	5·590	5·814	6·037	6·261	6·484
3	6·708	6·932	7·155	7·379	7·602	7·826	8·050	8·273	8·497	8·720
4	8·944	9·168	9·391	9·615	9·838	10·062	10·286	10·509	10·733	10·956
5	11·180	11·404	11·627	11·851	12·074	12·298	12·522	12·745	12·969	13·192
6	13·416	13·640	13·863	14·087	14·310	14·534	14·758	14·981	15·205	15·428
7	15·652	15·876	16·099	16·323	16·546	16·770	16·994	17·217	17·441	17·664
8	17·888	18·112	18·335	18·559	18·782	19·006	19·230	19·453	19·677	19·900
9	20·124	20·348	20·571	20·795	21·018	21·242	21·466	21·689	21·913	22·136
10	22·360	22·584	22·807	23·031	23·254	23·478	23·702	23·925	24·149	24·372
11	24·596	24·820	25·043	25·267	25·490	25·714	25·938	26·161	26·385	26·608
12	26·832	27·056	27·279	27·503	27·726	27·950	28·174	28·397	28·621	28·844
13	29·068	29·292	29·515	29·739	29·962	30·186	30·410	30·633	30·857	31·080
14	31·304	31·528	31·751	31·975	32·198	32·422	32·646	32·869	33·093	33·316
15	33·540	33·764	33·987	34·211	34·434	34·658	34·882	35·106	35·329	35·553

TABLE III.

Upstream slope = 2½ to 1; S = 2·693.

FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·269	0·539	0·808	1·077	1·346	1·616	1·885	2·154	2·424
1	2·693	2·962	3·232	3·501	3·770	4·039	4·309	4·578	4·847	5·117
2	5·386	5·655	5·925	6·194	6·463	6·732	7·002	7·271	7·540	7·810
3	8·079	8·348	8·618	8·887	9·156	9·425	9·695	9·964	10·233	10·503
4	10·772	11·041	11·311	11·580	11·849	12·118	12·388	12·657	12·926	13·196
5	13·465	13·734	14·004	14·273	14·542	14·811	15·081	15·350	15·619	15·889
6	16·158	16·427	16·697	16·966	17·235	17·504	17·774	18·043	18·312	18·582
7	18·851	19·120	19·390	19·659	19·928	20·197	20·467	20·736	21·005	21·275
8	21·544	21·813	22·083	22·352	22·621	22·890	23·160	23·429	23·698	23·968
9	24·237	24·506	24·776	25·045	25·314	25·583	25·853	26·122	26·391	26·661
10	26·930	27·199	27·469	27·738	28·007	28·276	28·546	28·815	29·084	29·354
11	29·623	29·892	30·162	30·431	30·700	30·969	31·239	31·508	31·777	32·047
12	32·316	32·585	32·855	33·124	33·393	33·662	33·932	34·201	34·470	34·740
13	35·009	35·278	35·548	35·817	36·086	36·355	36·625	36·894	37·163	37·433
14	37·702	37·971	38·241	38·510	38·779	39·048	39·318	39·587	39·856	40·126
15	40·395	40·664	40·934	41·203	41·472	41·741	42·011	42·280	42·549	42·819
16	43·088	43·357	43·627	43·896	44·165	44·434	44·704	44·973	45·242	45·512
17	45·781	46·050	46·320	46·589	46·858	47·127	47·397	47·666	47·936	48·205
18	48·474	48·743	49·013	49·282	49·551	49·820	50·090	50·359	50·628	50·898
19	51·167	51·436	51·706	51·975	52·244	52·513	52·783	53·052	53·321	53·591
20	53·860	54·129	54·399	54·668	54·937	55·206	55·476	55·745	56·014	56·284
21	56·553	56·822	57·092	57·361	57·630	57·899	58·169	58·438	58·707	58·977
22	59·246	59·515	59·785	60·054	60·323	60·592	60·862	61·131	61·400	61·670
23	61·939	62·208	62·478	62·747	63·016	63·285	63·555	63·824	64·093	64·363
24	64·632	64·901	65·171	65·440	65·709	65·978	66·248	66·517	66·786	67·056
25	67·325	67·594	67·864	68·133	68·402	68·671	68·941	69·210	69·479	69·749

TABLE IV.

Upstream slope = 3 to 1; S = 3·162.

FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
0	0·000	0·316	0·632	0·949	1·265	1·581	1·897	2·213	2·530	2·846
1	3·162	3·478	3·794	4·111	4·427	4·743	5·059	5·375	5·692	6·008
2	6·324	6·640	6·956	7·273	7·589	7·905	8·221	8·537	8·854	9·170
3	9·486	9·802	10·118	10·435	10·751	11·067	11·383	11·699	12·016	12·332
4	12·648	12·964	13·280	13·597	13·913	14·229	14·545	14·861	15·178	15·494
5	15·810	16·126	16·442	16·759	17·075	17·391	17·707	18·023	18·340	18·656
6	18·972	19·288	19·604	19·921	20·237	20·553	20·869	21·185	21·502	21·818
7	22·134	22·450	22·766	23·083	23·399	23·715	24·031	24·347	24·664	24·980
8	25·296	25·612	25·928	26·245	26·561	26·877	27·193	27·509	27·826	28·142
9	28·458	28·774	29·090	29·407	29·723	30·039	30·355	30·671	30·988	31·304
10	31·620	31·936	32·252	32·569	32·885	33·201	33·517	33·833	34·150	34·466
11	34·782	35·098	35·414	35·731	36·047	36·363	36·679	36·995	37·312	37·628
12	37·944	38·260	38·576	38·893	39·209	39·525	39·841	40·157	40·474	40·790
13	41·106	41·422	41·738	42·055	42·371	42·687	43·003	43·319	43·636	43·952
14	44·268	44·584	44·900	45·217	45·533	45·849	46·165	46·481	46·798	47·114
15	47·430	47·746	48·062	48·379	48·696	49·011	49·327	49·643	49·960	50·276
16	50·592	50·908	51·224	51·541	51·857	52·173	52·489	52·805	53·122	53·438
17	53·754	54·070	54·386	54·703	55·019	55·335	55·651	55·967	56·284	56·600
18	56·916	57·232	57·548	57·865	58·181	58·497	58·813	59·129	59·446	59·762

TABLE IV.—*continued.*

FEET.

Height in Feet.	Decimals.									
	0·0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9
19	60·078	60·394	60·710	61·027	61·343	61·659	61·975	62·291	62·606	62·924
20	63·240	63·556	63·872	64·189	64·505	64·821	65·137	65·453	65·770	66·086
21	66·402	66·718	67·034	67·351	67·667	67·983	68·299	68·615	68·932	69·248
22	69·564	69·880	70·196	70·513	70·829	71·145	71·461	71·777	72·094	72·410
23	72·726	73·042	73·358	73·675	73·991	74·307	74·623	74·939	75·256	75·572
24	75·888	76·204	76·520	76·837	77·153	77·469	77·785	78·101	78·418	78·734
25	79·050	79·366	79·682	79·999	80·315	80·631	80·947	81·263	81·580	81·896
26	82·212	82·528	82·844	83·161	83·477	83·793	84·109	84·425	84·742	85·058
27	85·374	85·690	86·006	86·323	86·639	86·955	87·271	87·587	87·904	88·220
28	88·536	88·852	89·168	89·485	89·801	90·117	90·433	90·749	91·066	91·382
29	91·698	92·014	92·330	92·647	92·963	93·279	93·596	93·911	94·228	94·544
30	94·860	95·176	95·492	95·809	96·125	96·441	96·757	97·073	97·390	97·706
31	98·022	98·338	98·654	98·971	99·287	99·603	99·919	100·235	100·552	100·868
32	101·184	101·500	101·816	102·133	102·449	102·765	103·081	103·397	103·714	104·030
33	104·346	104·662	104·978	105·295	105·611	105·927	106·243	106·559	106·876	107·192
34	107·508	107·824	108·140	108·457	108·773	109·089	109·405	109·721	110·038	110·354
35	110·670	110·986	111·302	111·619	111·935	112·251	112·567	112·883	113·200	113·516
36	113·832	114·148	114·464	114·781	115·097	115·413	115·729	116·045	116·362	116·678
37	116·994	117·310	117·626	117·943	118·259	118·575	118·891	119·207	119·524	119·840
38	120·156	120·472	120·788	121·105	121·421	121·737	122·053	122·369	122·686	123·002
39	123·318	123·634	123·950	124·267	124·583	124·899	125·215	125·531	125·848	126·164
40	126·480	126·796	127·112	127·429	127·745	128·061	128·377	128·693	129·010	129·326
41	129·642	129·958	130·274	130·591	130·907	131·223	131·539	131·855	132·172	132·488
42	132·804	133·120	133·436	133·753	134·069	134·385	134·701	135·017	135·334	135·650
43	135·966	136·282	136·598	136·915	137·231	137·547	137·863	138·179	138·496	138·812
44	139·128	139·444	139·760	140·077	140·393	140·709	141·025	141·341	141·658	141·974
45	142·290	142·606	142·922	143·239	143·555	143·871	144·187	144·503	144·820	145·136
46	145·452	145·768	146·084	146·401	146·717	147·033	147·349	147·665	147·982	148·298
47	148·614	148·930	149·246	149·563	149·879	150·195	150·511	150·827	151·144	151·460
48	151·776	152·092	152·408	152·725	153·041	153·357	153·673	153·989	154·306	154·622
49	154·938	155·254	155·570	155·887	156·203	156·519	156·835	157·151	157·468	157·784
50	158·100	158·416	158·732	159·049	159·365	159·681	159·997	160·313	160·630	160·946
51	161·262	161·578	161·894	162·211	162·527	162·843	163·159	163·475	163·792	164·108
52	164·424	164·740	165·056	165·373	165·689	166·005	166·321	166·637	166·954	167·270
53	167·586	167·902	168·218	168·535	168·851	169·167	169·483	169·799	170·116	170·432
54	170·748	171·064	171·380	171·697	172·013	172·329	172·645	172·961	173·278	173·594
55	173·910	174·226	174·542	174·859	175·175	175·491	175·807	176·123	176·440	176·756
56	177·072	177·388	177·704	178·021	178·337	178·653	178·969	179·285	179·602	179·918
57	180·234	180·550	180·866	181·183	181·499	181·815	182·131	182·447	182·764	183·080
58	183·396	183·712	184·028	184·345	184·661	184·977	185·293	185·609	185·926	186·242
59	186·558	186·874	187·190	187·507	187·823	188·139	188·455	188·771	189·088	189·404
60	189·720	190·036	190·352	190·669	190·985	191·301	191·617	191·933	192·250	192·566
61	192·882	193·198	193·514	193·831	194·147	194·463	194·779	195·095	195·412	195·728
62	196·044	196·360	196·676	196·993	197·309	197·625	197·941	198·257	198·574	198·890
63	199·206	199·522	199·838	200·155	200·471	200·787	201·103	201·419	201·736	202·052
64	202·368	202·684	203·000	203·317	203·633	203·949	204·265	204·581	204·898	205·214
65	205·530	205·846	206·162	206·479	206·795	207·111	207·427	207·743	208·060	208·376
66	208·692	209·008	209·324	209·641	209·957	210·273	210·589	210·905	211·222	211·538
67	211·854	212·170	212·486	212·803	213·119	213·435	213·751	214·067	214·384	214·700
68	215·016	215·332	215·648	215·965	216·281	216·597	216·913	217·229	217·546	217·862
69	218·178	218·494	218·810	219·127	219·443	219·759	220·075	220·391	220·708	221·024
70	221·340	221·656	221·972	222·289	222·605	222·921	223·237	223·553	223·870	224·186
71	224·502	224·818	225·134	225·451	225·767	226·083	226·399	226·715	227·032	227·348
72	227·664	227·980	228·296	228·613	228·929	229·245	229·561	229·877	230·194	230·510
73	230·826	231·142	231·458	231·775	232·091	232·407	232·723	233·039	233·356	233·672
74	233·988	234·304	234·620	234·937	235·253	235·569	235·885	236·201	236·518	236·834
75	237·150	237·466	237·782	238·099	238·415	238·731	239·047	239·363	239·680	239·996
76	240·312	240·628	240·944	241·261	241·577	241·893	242·209	242·525	242·842	243·158
77	243·474	243·790	244·106	244·423	244·739	245·055	245·371	245·687	246·004	246·320
78	246·636	246·952	247·268	247·585	247·901	248·217	248·533	248·849	249·166	249·482
79	249·798	250·114	250·430	250·747	251·063	251·379	251·695	252·011	252·328	252·644
80	252·960	253·276	253·592	253·909	254·225	254·541	254·857	255·173	255·490	255·816

APPENDIX 22.

NOTES ON THE ARRANGEMENTS FOR AND MANAGEMENT OF LARGE WORKS.

(*Vide* Chapter II., paragraph 126 (*c*).)

INDEX TO PARAGRAPHS.

1. *Work Arrangements.*

Para.

1. General Arrangements for Works.
2. Setting-out Marks.
3. Side-widths and Levels.
4. Bench Marks.
5. Setting-out Pillars.
6. Foundation Plan.
7. Foundation Courses.
8. Setting-out Platforms.
9. Progress Sections.
10. Programme of Work.
11. Temporary Works.
12. Rapid Completion of small Works.
13. Works Roads.
14. Tidying up Works.
15. Water.
16. Bailing.
17. Reserve Works.
18. Preparation for the Monsoon.
19. Works Order-book.
20. Maistries' Order-book.
21. Maistries' Appliances.
22. Hand Sketches.
23. Supervision of Construction.
24. Boundaries of Acquired Land.

Para.

25. Survey Marks Affected by Works.
26. Excavating for Materials in Land Temporarily Acquired.
27. Native Holidays.
28. Works Postal Arrangements.

2. *Miscellaneous.*

29. Sanitation.
30. Blasting.
31. Serious Accidents.

3. *Materials.*

32. Material Requirements.
33. Materials at Site.
34. Collection of Materials.
35. Purchase of Materials.
36. Selection of Materials.
37. Issue of Materials.
38. Balances of Materials at Site.
39. Protection of Materials.
40. Lime-burning Account.
41. Gelatine and Dynamite, &c., Cartridges.
42. Powder.

4. *Stores and Tools.*

43. Stores and Workshops.
44. The Store.

Para.
45. Stamping Tools.
46. Register of Tools.
47. Issue of Tools to Contractors.
48. Issue of Plant to Contractors.
49. Issue of Tools to Departmental Gangs.
50. Returning Tools, &c., out of use.
51. Broken Articles.
52. Watering Pots.
53. Rammers.
54. Miscellaneous Articles.
55. Setting-out Materials.
56. Indents for following day's Requirements.
57. Register of Expenditure of "Sundries."
58. Receipts and Issues.

5. *Plant.*

59. Rules to be Observed in the Use of Tram Plant.
60. Supervision of Pumping and other Engines.
61. Pumping Engines and Special Plant.
62. Pumps.
63. Boat.
64. Stacking Woodwork.
65. Removal of Plant and Material before Monsoon.

6. *Establishment.*

66. Sub-division of Works.
67. Duties of Establishment.
68. Appointment of Work Establishment.
69. Appointments on Works.

Para.
70. Muccadums' Pay.
71. Muccadums' Badges.
72. Independence of Maistries and Karkuns.
73. Authority of Office Establishment on Works.
74. Daily Inspections.

7. *Labour.*

75. Working Hours.
76. Off Time.
77. Rates and Numbers of Labourers.
78. Tasking Work.
79. Distribution of Work.

8. *Petty Contractors.*

80. Agreements with Petty Contractors and Suppliers.
81. Petty Supply Contract.
82. Rates of Supply by Petty Contractors.
83. Supervision of Petty Contractors' Work, &c.
84. Number of Petty Contractors' Labourers.
85. Fining Contractors' Workpeople.

9. *Office Arrangements.*

86. Sub-divisional Office Duties.
87. Sub-divisional Returns.
88. Final Measurements.
89. Bills and Measurement Books.
90. Daily Measurements of Work.
91. Final Measurements of Work and Supplies.
92. Daily Report.
93. Weekly Report.

I. Work Arrangements

1. General Arrangements for Works.—Before the works are commenced, a large scale plan should be made showing how the different offices, temporary works, (such as kilns and mortar mills), in connection with manufacturing departments, and stores of materials are to be arranged most conveniently for the whole of the works. A liberal amount of space should be allotted to each.

The main office, the storeyard, workshop and establishment quarters should be together in a central situation removed from the village, &c.

The material stacking-ground should be arranged so that the rough materials should be furthest from and the finished ones nearest to the works. Thus for mortar the order would be: Sand, kankar, fuel, kilns and mortar mills. It should be divided into two parts, one from which supplies are being drawn, and the other on to which they are being delivered. The former should be quite cleared before fresh supplies are brought on to it.

All should be set out so as not to interfere with the works from their commencement up to their completion.

2. Setting-out Marks.—At the ends of all principal lines of the works should be erected small masonry pillars, each having a slab, at the top of which the centre line and Reduced Level should be engraved. Distance marks should be fixed at right angles and opposite to the end of each chain at a specified distance from the centre line, and their chainage should be engraved on them. As far as possible all such marks should be fixed permanently and out of the way during construction. The toe-lines of all embankments and the centre lines of all works should be lockspitted before they are begun.

3. Side-widths and Levels.—Tables of these for each part of the work should be prepared, and copies of them should be given to all who have to set out the work.

4. Bench Marks.—These should be left as frequently as possible, so that intermediate readings need never be necessary, and should be of a permanent nature. A list of them should be kept by all levellers at the beginning of their note-books.

Permanent marks on the works should be dressed reversed-V shape in section and leaded.

5. Setting-out Pillars.—These should be constructed similarly, so as to be of permanent utility.

6. Foundation Plan.—A large scale plan of the foundations should be prepared, and all levels should be carefully recorded on it. It will

be best to divide the area into small rectangular compartments and to take the mean level of each, noting any sudden changes and the quantity of filling required to make up the work to the top or to any assumed level.

7. Foundation Courses.—Particular care is necessary to set these out to the correct widths and levels. On no account should one course be erroneously run into another. On each should be painted its reference level. Teak battens should be fixed parallel to the face of the work as profiles, and references to the proper position of the top edges of the courses should be marked on them.

8. Setting out Platforms.—Archrings should be set out full size on plastered platforms, each voussoir being drawn thereon. Similarly, curved profiles, cut-water courses, caps, &c., should be drawn out full size.

It will assist the workmen if full-sized models of difficult parts of the work are made.

9. Progress Sections.—A longitudinal section and cross sections of progress effected should be carefully maintained. The top of the work as completed each month should be carefully levelled and plotted thereon, and each month's work should be indicated with distinctive colour. On the cross sections levels should be recorded. Separate sections should be maintained for excavation and filling.

10. Programme of Work.—As soon as the works are fairly started a programme of anticipated progress should be made. This should take into account the times by which certain stages must be completed, and must allow liberally for all contingencies. The actual progress of the work should be kept in advance of it, to allow for possible future delays.

11. Temporary Works.—These should be invariably set out neatly to proper lines and levels. Profiles of these works should be erected in advance whenever possible, so that their future positions and sizes may be seen at a glance.

12. Rapid Completion of Small Works.—Small works, repairs, &c., specially ordered by the Officer, should be completed within three days of his order, so that further reference to them will be unnecessary. They should take precedence of the general routine work, and additional labourers (carpenter, blacksmith, &c.) should be engaged, if necessary, to complete them.

13. Works Roads.—All roads should be set out outside the permanent works, and should be arranged to be carried up with them as they rise. They should, as a rule, be executed departmentally, and not

by contractors. No materials should be stacked on them. If practicable, they should be arranged so as to be permanently useful after the completion of the works.

14. Tidying up Works.—The works must be always left in a tidy state at night, *i.e.*, all mortar should be used up or stacked; all concrete thoroughly rammed; and watering cans, rammers, &c., returned to store. The Sub-Divisional Officer must himself see to this. No rubbish, loose stones, &c., should be allowed on the works or works roads; these can be cleared off by children.

15. Water.—Arrangements should be made for water to be cheaply available at all parts of the work by means of pipes, pumps and hose, bhistees, cisterns, &c. It is essential that ample water is available throughout the fair season, and, if necessary, small storage reservoirs should be formed immediately at the end of the monsoon by damming nullas, &c.

16. Bailing.—For lifts up to 3 feet, and where there is plenty of room, the basket "súp" is the best means of bailing. It can be also used with double shifts up to 6 feet. Beyond that height, large or small pumps should be used. Bailing need not be done, as a rule, where the water level is below that of the blast holes.

17. Reserve Works.—Certain work should be reserved for execution during rainy days or when the wetness of the ground will prevent the continuance of the main work. Preparation of concrete metal is a good work for this purpose.

18. Preparation for the Monsoon.—All plant and material liable to be submerged should be removed to places of safety in ample time. All burnt kankar should be slaked and the lime screened and stored. All arrangements should be made beforehand for moving material to where it will be required, as this can be done most cheaply and cleanlily in the fair weather.

19. Works Order-book.—A book should be opened in which all important orders are to be recorded. Each order should be numbered for reference and dated, and the orders should be separated into distinctive sections. In a marginal column should be entered the date of completion of the order and any additions to or modifications of it found necessary. These latter should be made in red ink, so as to catch the eye. A copy of the book should be kept by each Subordinate, who should send it daily to the office at 2 p.m. to have new entries made in it.

The orders given in this book should have all the force of those separately and specially communicated by letters, and they must be


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carried out as quickly as possible. The date of completion of an order should be written in the right-hand column, and any information which can be concisely given should be also entered there.

20. Maistries' Order-book.—A rough note-book should be given to the Maistry, and in it extracts from the general works order-book which apply to his charge should be posted by the subordinate in vernacular. Blank pages should be left for the Officer to write his orders on the spot; these should be shown at once by the Maistry to the Subordinate, and should be translated by him. Important orders should be given in writing, not verbally, otherwise they are liable to misinterpretation. Verbal orders should not be accepted as authority for any change in the work.

21. Maistries' Appliances.—Each Maistry should be provided with setting-out appliances, *i.e.*, one 50-feet tape (common), mason's level, plumb-line, foot-rule, string, chalk, &c. A small boy may be told off to each Maistry to hold these, to run messages, &c. Muccadums may be given string and chalk.

22. Hand Sketches.—Dimensioned hand sketches should be always given to those directly in charge of any minor work, and these should be kept on it for ready reference.

23. Supervision of Construction.—Unless in exceptional circumstances all construction work should be under the direct supervision of a Maistry, and should not be left to a Muccadum alone.

24. Boundaries of Acquired Land.—These should be permanently marked, and at once.

25. Survey Marks affected by Works.—A list of these should be sent at once to the Revenue authorities, with a view to the correction of their records, and their action in respect to their replacement awaited as long as possible.

Off-sets to fixed points should be taken from each, so that their original positions may be determined if their removal is urgent.

26. Excavating for Materials in Land Temporarily Acquired.—When land has to be returned to the owners, the top soil should be carefully removed and stacked separately, so that, after the required material (such as kankar) has been removed, this soil may be replaced so as to permit of the cultivation of the area. For the same reason the area should be left levelled and tidy and free from stones, rubbish, &c.

27. Native Holidays.—One week's notice should be given to the Officer of all holidays which are likely to interfere with the progress of the work, so that due arrangements may be made.

28. Works Postal Arrangements. — Each Subordinate and Maistry should be accompanied on the works by a small boy who can take messages. In addition there should be a man who can take and deliver all official papers, &c., in a leather bag or tin case to all sections of the work.

2. Miscellaneous.

29. Sanitation.—(*a*) No one should be allowed to go for relief or to wash clothes, bathe, eat, &c., above, or at, the source of water-supply.

(*b*) No one should resort for relief to within 100 feet of nulla and river beds.

(*c*) If the people will resort to trenches, these should be dug in neat lines and filled with earth each day, fresh ones first being prepared by sweepers.

(*d*) If they will not do this, specified squatting areas, one for men, the other for women, should be marked out by yellow flags at reasonable and convenient distances from the work, where they are not likely to produce nuisance.

(*e*) Piecework gangs and all who camp on the works must keep clean their surroundings for 200 yards, or be made to live further off.

(*f*) Sanitary guards, with numbered yellow belts, should be appointed to see that these orders are carried out. Also, a few sweepers should in any case be engaged.

(*g*) Specified watering-places, or wells, for each caste should be arranged for; it will save waste of time if cylinders of water are kept for them under guard on the works themselves. One or more buckets should be specially reserved for each watering-place, and others should not be used.

(*h*) A separate set of water guards, with numbered blue belts, should be appointed.

(*k*) A responsible karkun, &c., should be appointed to superintend the sanitary guards and to report the names of all offenders.

30. Blasting.—Every precaution should be taken to avoid accidents.

(*a*) A responsible man should be put in charge.

(*b*) Blasting, as a rule, should take place at fixed times only, *e.g.*, at the mid-day meal and at sunset.

(*c*) Only copper tamping bars and needles should be used, and they should be kept bright for the easy recognition of their material.

(*d*) A cordon of watchmen with red flags should be formed around the area of the blasts, and 300 feet from them. The flags should be shown two minutes before blasting begins and until the last blast has gone off. In case all the blasts do not explode, the flags should not be lowered until ten minutes after the last explosion.

(*e*) The number of blasts at one time and place should be limited to six, and these should be counted and recorded by the man-in-charge.

(*f*) Fresh blasting should not be carried out without the orders of the man-in-charge.

(*g*) Fuzes must be cut to the proper length *before* the holes are filled with powder and tamped.

31. Serious Accidents.—The sufferers should be at once taken to the works hospital, and the medical man summoned. The Officer and his Subordinates should be informed as soon as possible.

3. Materials.

32. Material Requirements.—The quantities of materials required for a given amount of work should be estimated from time to time, so that stocks of different classes may be regulated to requirements.

33. Materials at Site.—Sufficient material of all descriptions should be collected and kept available so as to last at least for one month, and preferably for two months. This will prevent strikes for higher rates among suppliers and the stoppage of any work from deficiency of supply.

34. Collection of Materials.—A special man, not under the rank of Sub-overseer, should be told off to superintend the suppliers of materials, many of whom will be working some distance from the works. He should see that the material is up to specification and is being supplied according to ordered quantities. He should separate rejected materials from the rest before measurements are taken, and should, as far as possible, prevent them from being brought to site. He should meet the Officer once every second day to keep him informed of what is being done. Lists of special requirements should be given to him from time to time.

35. Purchase of Materials.—Materials such as sand, slaked lime, kankar, charcoal and metal should be stacked under departmental supervision in heaps of specified height, so as to obtain fair measure. Rubble stones should be bought in heaps of specified dimensions, and a specified deduction made therefrom to allow for vacuities. The method of stacking should be carefully watched to prevent fraud.

36. Selection of Materials.—A special and well-qualified man should be kept to see that all stones, metal, and other materials are of the specified size, quality, &c., before they are stacked for measurement. Those which are not, should be marked at once with a distinctive mark, stacked separately for inspection, and removed from the works as soon as they have been finally rejected by the Officer or Subordinate.

37. Issue of Materials.—No material should be issued to anyone except by the Maistry in charge, who should account for the same to the Subordinate daily.

38. Balances of Materials at Site.—These should be actually measured by the close of the works month, as on them the proper calculation of the cost of the work depends. They should be reported on the second day of the new month by the Subordinate in charge of materials to the Officer. To facilitate this measurement, all paid-for heaps should have on top flat reference stones, with their cubic capacity painted thereon. All these stones should be in place before the original measurements are taken. The quantity of stock in hand can thus be readily ascertained at any time.

39. Protection of Materials.—Due precautions should be invariably taken to shelter all materials requiring protection from sun and rain, and to guard carefully all costly and readily saleable material. All wooden articles should be stacked on stones off the ground.

40. Lime-burning Account.—Accounts of the cost of manufacture and the amount used on the works should be carefully kept. Also, the out-turn of each kiln should be recorded in a register.

Where cement is used, a register of the issue of barrels should be kept, and the empty ones returned to store.

41. Gelatine and Dynamite, &c., Cartridges.—These cartridges should be kept in a separate magazine away from the works. Their detonators and fuze should be kept under lock in the store. The cartridges should be issued twice a day in time to allow of their being primed, and, until handed over to the blasters, must be in the charge of a responsible man. The junction of the fuze and the detonator should, when necessary, be made water-tight with wax, tarred over. A careful register of issues, blasts, and returned cartridges should be kept. All unexploded cartridges should be at once returned to the magazine after the primers have been removed.

42. Powder.—Blasting powder should be stored in wooden casks buried under ground.

4. Stores and Tools.

43. Stores and Workshops.—A special sub-charge should be made of these. In regard to stores, the different sub-divisional officers should frame monthly indents on the chief Storekeeper so as to replenish their own stores, and these should be sanctioned by the head of the works.

Large repairs should be sent to the workshop, small ones being effected by the sub-divisional staff.

44. The Store.—Each class of store should be kept separate from the rest, and all outside stores should be raised from the ground.

45. Stamping Tools.—All tools in use should be stamped. Each letter will cost from two to three annas per hundred tools.

46. Register of Tools.—In this one or more pages should be devoted to each kind of tool in ordinary, general use. In it the dated issues to (in black) and returns (in red) by each petty contractor and sub-division should be shown by the Storekeeper, so that he can readily ascertain and check his balance at any time.

47. Issue of Tools to Contractors.—A register of all issues and receipts should be kept, with a separate page, or pages, for each Contractor. Persons supplied should sign each entry, and afterwards the Subordinate should ascertain from them that they have received the full number entered against their names, and should also initial the entry. Each entry should give the number, size, weight, condition, &c., of the articles issued.

48. Issue of Plant to Contractors.—Where there are several Contractors on a work, a register of small plant, such as trucks, measures for concrete, &c., ladders and screens, &c., should be maintained, and the articles should be marked temporarily with the Contractors' initials.

49. Issue of Tools to Departmental Gangs.—Tools may be similarly issued to Muccadums of gangs, and may be kept with them, provided that on the days on which musters are closed all such tools are counted by one of the staff. Missing tools should have their value recovered from the defaulters at the time of payment.

50. Returning Tools, &c., out of use.—All tools and articles of plant no longer required on the work should be at once returned to store.

51. Broken Articles.—All such as cannot be mended on the works themselves should be at once sent to the workshop, and should be repaired at once. If unrepairable, they should be at once weeded out and kept separately in the main store for final disposal.

52. Watering Pots.—Galvanised watering pots, being costly, should be issued only to special men, who should be held responsible for their damage or loss. Common ones made of old kerosine tins should be generally employed. For masonry work the roses should be soldered on to prevent their loss. Roses are only necessary for watering metal to be mixed for concrete, and for concrete and masonry less than two days old; other watering should be done plentifully by buckets.

53. Rammers.—Wooden rammers should be made of "babhul," or hard wood, 7 inches to 8 inches diameter at base, 5 inches to 6 inches

at top, and 8 inches high. The handles of these and other tools are best made of "babhul," or tough wood; their ends should be split, and they should be wedged up.

54. Miscellaneous Articles.—A sufficient stock of all miscellaneous articles required on the works, *e.g.*, mortar mill poles and centre posts, mortar millstones, scrapers, roller poles, ladders, boning rods, handles, rammers, wooden lime rakes, templates, tramway fittings, lime, cement and metal measures, &c., should be always kept ready. A list of the stock required should be made out and kept for guidance in the workshop.

A list of fittings and bolts, showing the amount of stock required to be maintained, balance on works and balance to be procured, should be submitted monthly. As soon as any stock article is issued another should be made to replace it.

Similarly, a month's supply of all petty stores should be kept by the Storekeeper, and should be replenished by means of monthly indents on the dealers.

55. Setting-out Materials.—These should be invariably kept in readiness in the sub-divisional store. They comprise bamboos, pegs, (large and small,) string, coir rope, chalk, white, black and red paint, tar, brushes, plumb-bobs, mason's squares, mason's levels, teak templates, &c.

56. Indents for Following Day's Requirements.—Each Subordinate should prepare this by 2 p.m., and get the Officer's sanction, so that it may be issued from the store in time. Dimensions of articles and sketches should be given when necessary, and the indent should be arranged under heads in accordance with the separate minor works for which the articles are required.

57. Register of Expenditure of "Sundries."—This should be kept regularly and posted daily, each entry being initialled by the officer in charge to see that expenditure is properly controlled and debited.

58. Receipts and Issues.—All articles and materials sent from one part of the works to another should be accompanied by vouchers, or acknowledgments, which should be obtained and recorded so as to prevent mistakes occurring.

5. Plant.

59. Rules to be Observed in the Use of Tram Plant.—(*a*) Immediately a truck is in need of repair it should be taken off the rails and placed neatly on one side until it is repaired.

(*b*) Truck men should not run with their trucks, but should proceed quietly.

(*c*) Trucks should not be loaded above their tops.

(*d*) If a loaded truck leaves the rails, it should be emptied before it is *lifted* on. Bars should never be used for levering trucks on to the rails.

(*e*) All axles should be regularly oiled and a piece of oiled waste kept in the pedestal.

(*f*) The tram line should be laid in regular lines and curves, with as few ups and downs as possible. It should be kept well ballasted, and all ties and joints, &c., should be constantly kept in repair. All switches and points should be kept free of sand, dirt, &c. Where field drainage is crossed, the line should be laid on single stone sleeper piers.

(*g*) All trucks and carriages should be numbered. They should be examined each evening in the presence of their users near the sub-divisional office; all missing parts should be noted, and they should be replaced at the cost of the users. The trucks should be then oiled ready for next day's work.

(*h*) A special gang should be kept for tram laying, &c. When the line has been once well laid, a very small number of men will suffice for this purpose.

(*k*) All repairs should be done by skilled blacksmiths.

(*l*) To avoid confusion, trucks should go in "trains," *i.e.*, a number together.

(*m*) All loose parts of the tramway and trucks should be neatly stacked near the workshop or store.

(*n*) All trucks should be examined once a week by the Sub-divisional Officer himself.

(*o*) A standard gauge for rail laying should be made and given to the rail layers.

(*p*) The number of serviceable trucks should be noted in the weekly report.

(*r*) As a rule trams should not be used at night, or when there is not departmental supervision.

60. Supervision of Pumping and other Engines.—These must be invariably placed under the charge of a thoroughly competent and certificated man. He should be supplied with the best materials and appliances, including special dubbin for belts. He should be held solely responsible for everything under his charge. All engines should be thoroughly cleaned up, washed out and repaired once a week.

61. Pumping Engines and Special Plant.—These should be carefully and thoroughly inspected once a week by the Subordinate in charge of the workshop. Petty repairs should be done by skilled men only, and all materials used should be of the best quality. Such plant should be protected from the weather, and should be cleaned and painted before the rains.

62. Pumps.—All valve leathers should be kept moist continuously. All valves should be kept in a state of repair. Hose should be repaired as soon as this becomes necessary; should be carefully coiled when out of use; and should be lifted, not dragged, over rough ground. Suction hose should be wrapped round with thin coir rope to protect it from abrasion and the action of light.

63. Boat.—This should be kept under cover as much as possible, and should be loaded when it is out of use, so as to keep under water all planks liable to be submerged when it is in use, and thus to prevent their cracking. The lower sides of gratings should be tarred and the upper sides oiled. All unpainted wood should be oiled and brass work kept bright. The leather buttons on the bars should be greased with fat, and the oars stacked on level planks to prevent them warping. The inside of the boat should be kept dry. Coir rope fenders should be provided to protect the sides of the boat from injury.

In tanks, boats should be anchored thus: An old millstone, or other heavy weight, should be placed on the tank bed, and to it should be fastened a chain which is attached at the other end to a buoy, to which the boat itself should be fastened.

64. Stacking Woodwork.—All wood and wooden articles out of use should be neatly stacked under shade and raised from the ground on stones.

65. Removal of Plant and Material before Monsoon.—All plant and material liable to be submerged, or to be carried away, should be removed to places of safety by May 15th. No fresh material should be thereafter stacked where it is liable to suffer damage.

6. Establishment.

66. Sub-division of Works.—On a large work each Upper Subordinate should have a definite charge, and should have special establishment and a small store allotted to him.

67. Duties of Establishment.—The duties of each member should be laid down in writing, and each should be held wholly and solely responsible for his definite charge.

68. Appointment of Work Establishment.—This should be done according to some scale thus:—

1 Maistry to 50 masons;

1 Karkun to 200 labourers;

1 Muccadum to 50 labourers.

These are in addition to the general supervising staff.

69. Appointments on Works.—All appointments should be made by the Officer of highest rank on the work, or be sanctioned in writing by him. Also, all changes in pay and classification should be similarly sanctioned.

70. Muccadums' Pay.—The pay of these men should bear some relation to the number of people they bring on to the works. They should be held responsible for keeping up the strength of their gangs, and for giving early information about those who intend to leave or have left the works.

71. Muccadums' Badges.—These should consist of red cloth, 3 inches in diameter, on which distinctive numbers, about 2 inches high, should be printed in black, so that the men can be identified at any time. The badges should be sewn on to their left sleeves.

72. Independence of Maistries and Karkuns.—Maistries should not have control over Karkuns, and *vice versâ*. Each should be quite independent of the other.

73. Authority of Office Establishment on Works.—Clerks and peons should not have authority on works, and coolies and materials should not be given to them without written orders.

74. Daily Inspections.—The Maistries should accompany the Officer when he commences his inspection, and the Subordinate, on his return journey. All orders for work and complaints should be disposed of by the Officer with the latter at the time, and labourers should be thus informed.

7. Labour.

75. Working Hours.—The commencement and end of the working day should be announced by a horn. Half an hour after sunrise and sunset itself may be fixed for these limits. Mustering should begin at once, and late arrivals should be kept separate, so that they may be dealt with afterwards.

76. Off Time.—Specified times should be fixed for meals, and no one should be allowed to eat at other times. The commencement and end of "off time" should be announced by a horn, and a flag should be kept hoisted in a conspicuous place during its continuance. In cases

where rapid progress is necessary, continuous work should be arranged for by employing special gangs when the ordinary ones are off the work.

77. Rates and Numbers of Labourers.—A scale of the maximum and minimum rate of labour should be laid down. Thereafter the Subordinate-in-Charge may be authorised to fix the rates in individual cases. Similarly, he should fix the number required for any particular work.

Although Maistries and Karkuns may be consulted in such matters, they must not be allowed to fix rates and settle numbers.

78. Tasking Work.—Rates should be fixed for all classes of work, and the labourers kept up to their allotted tasks and paid accordingly. Due notice of their work should be given to them and its extent clearly explained to them. For certain classes of work the ticket system may be used, but quantities should be checked by tape measurements periodically.

79. Distribution of Work.—By 5 p.m. every one should know what and where is to be his work for the following day, so that then he may at once proceed to it. The Officer should give his orders to the Subordinates, these to the Maistries, and these to the gang Muccadums.

8. Petty Contractors.

80. Agreements with Petty Contractors and Suppliers.—These should be recorded in a book, each being signed or attested by the supplier, &c. They should state shortly :—

(*a*) The nature, specification, and quantity of supply, &c.;

(*b*) The rate ;

(*c*) The time by which quantities of supply, &c., are to be effected and measurements to be taken ;

(*d*) The penalties to be enforced for late, short, or inferior supplies, and for inattention to orders.

It is best to have agreements with single men, and not with partners. All agreements should be subject to the approval of the final sanctioning authority.

81. Petty Supply Contract.—A contract to last for one year should be entered into with a respectable dealer to supply all miscellaneous articles required on the works. Monthly indents prepared by the Store-keeper on receipt of the sub-divisional indents should be sent to him, and he should be bound to supply all articles within a fixed time.

82. Rates of Supply by Petty Contractors.—As soon as rates are settled, a list of them should be made out and posted, so that all intending

suppliers may know them. More liberal rates may be given at first to attract labour and material, and the dates up to which they will be in force should be intimated. After fair rates have been once settled, it is not advisable to make further alterations in them. Where the same materials have to be procured from different localities, a table of additional rates for carriage should be made out.

83. Supervision of Petty Contractors' Work, &c.—Reliable Muccadums should be told off to see that all such works are properly carried out.

Similarly, special men should be told off to every departmental or other operation, such as lime burning, sand washing, mortar mixing, &c.

84. Number of Petty Contractors' Labourers.—Each Maistry should keep a tabular register of the numbers of such labourers on each work under him, so that, in the case of dispute about the fairness of rates, the matter may be inquired into. The Subordinate should check these and abstract them in a book or register daily.

85. Fining Contractors' Workpeople.—A list of fines should be prepared monthly and the total recovered from the contractor, either by deduction from his bill or by ready-money payment. The list should be sanctioned by the Officer.

9. Office Arrangements.

86. Sub-divisional Office Duties.—The Sub-divisional Clerk should audit and check daily reports, muster rolls, bills, measurement books, and other cash vouchers and imprest accounts. He should prepare daily reports and other returns, day books, register of works, and all account papers and returns exclusive of those given to the Cashier.

The Second Clerk should register, copy, despatch, and file all correspondence.

The Cashier should complete measurement books, prepare receipts, bills, and imprest accounts, and compare correspondence. He should bring cash from the treasury, effect all payments, and be responsible for all cash and cash transactions. He should initial all cheques in token of having examined and found them correct. When making payments of muster rolls, he should count out only a little more money than is required, and, when the payment is finished, he should check the balance with the rolls before returning it to the cash bags.

The Storekeeper should prepare store accounts and indents for store supplies. He should maintain careful registers of all tools and plant and store issues. He should keep a sufficient balance of everything required

on the works, and make timely indents for all supplies of which the store is short.

87. Sub-divisional Returns.—These should be posted directly the entries can be made, so as to avoid a rush of work at the end of the month.

88. Final Measurements.—Before these are taken, all side widths and slopes should be properly taken out, and, before the bills are made out, all "deadmen" should be removed and the foundation cleared. Cross and longitudinal sections of completed foundations should be taken as soon as they are ready, and the length to which the cross section applies should be noted carefully.

89. Bills and Measurement Books.—All persons connected with the preparation or audit of these should enter their dated initials to the transaction concerned.

90. Daily Measurements of Work.—These are to be taken each evening with such accuracy that they will not differ more than 5 per cent. from the month's final measurements.

91. Final Measurements of Work and Supplies.—These should be recorded in measurement books, each chain of work being separately entered and its totals carried forward, so that a comparison with the estimate and a check can be made at any time.

Similarly, the measurements of each class of material should be separately recorded.

92. Daily Report.—This should be submitted each day before 2 p.m. on a printed form having columns for:—

The numbers of each principal class of labour, one being for establishment, on each subwork;

The approximate quantities of work done during the previous day;

Remarks about any particular circumstance, reasons for increase or decrease of labourers, or out-turn, &c.

93. Weekly Report.—The daily reports should be abstracted into a printed weekly report, which should record the daily numbers of different classes of labourers and the approximate total amount of work done. On the back should be entered general remarks noting:—

The levels to which the work was completed;

The quantity of work completed;

Reasons for slow or quick progress;

Any particular occurrences during the week.

The weekly reports should thus give a complete and fairly accurate history of the works.

APPENDIX 23.

TABLE[1] OF TASK-WORK IN DIFFERENT SOILS.

Explanatory Note.

The table has been drawn up for the purpose of determining and arranging in a systematic manner the quantities of work required as daily tasks from men, women, and children engaged upon the construction of earthwork, and has proved particularly useful where large bodies of workpeople, such as famine or extra-mural labour, have been employed.

The workpeople should be divided into gangs, each gang consisting of fifty people—men, women, and children—under one muccadum; six gangs being placed under one mustering karkun.

The following example illustrates the use of the Table :—

Assume a gang consisting of 20 men + 20 women + 10 children to be working with a lift varying from 6 to 10 feet, and a lead varying from 200 to 300 feet, in soft murum.

In the column including this lift and lead, the relative tasks for one man, one woman and one child are 48, 32 and 16 cubic feet respectively :

$$\begin{aligned} \text{Then } 48 \times 20 &= 960 = \text{task of 20 men.} \\ 32 \times 20 &= 640 = \text{,, ,, 20 women.} \\ 16 \times 10 &= 160 = \text{,, ,, 10 children.} \\ \hline & 1{,}760 \text{ cubic feet} = \text{total task} \end{aligned}$$

to be executed by the gang, if the material were black or red soil. As, however, the example refers to soft murum, the multiplier, 0·82 (see the footnote to the Table) must be used :

$$\text{Total task} = 1{,}760 \times 0{\cdot}82 = 1{,}443{\cdot}20 \text{ cubic feet,}$$

and this is given to the muccadum in round numbers. Thus, in the case of a puddle trench 10 feet in width, the task would be given as follows :—

Length.	Width.	Depth.
140 feet	× 10 feet	× 1 foot.

[1] Extracted from Marryat's "Specifications, Rates and Notes on Work," 4th ed. pp. 444—448. (Drawn up by Mr. C. T. Burke B.E., M.Inst.C.E.)

In the construction of a dam where the earth is put on in layers of uniform thickness, tasks should be measured on the dam.

For breaking lumps of earth, spreading and making the layers uniform, two men should be allowed for each gang—that is, one man for each twenty-five people employed.

In puddling, in layers of 1 foot in thickness, three men should complete, including cutting, treading and covering, a space of 25 feet × 10 feet, or 250 cubic feet in one day.

In "ramming" with a rammer weighing 30 lbs., one man should complete 500 square feet per day.

The following Tables have reference more especially to earthwork in large dams and embankments :—

TABLE SHOWING AN AVERAGE DAY'S WORK PER MAN, WOMAN, AND CHILD UNDER DIFFERENT "LIFTS" AND "LEADS" IN BLACK OR RED SOIL.

Lift, feet.		Lead, feet.		Average Day's Work, cubic feet.		
From	To	From	To	Man.	Woman.	Child.
...	5 inclusive.	...	50	76	51	25
		50	100	70	47	23
		100	200	64	43	21
		200	300	59	39	20
		300	400	53	35	18
		400	500	48	32	16
		500	600	42	28	14
		600	700	36	24	12
		700	800	30	20	10
		800	900	24	16	8
		900	1,000	18	12	6
5	10 inclusive.	...	100	56	37	19
		100	200	52	35	17
		200	300	48	32	16
		300	400	44	29	15
		400	500	40	27	13
		500	600	35	23	12
		600	700	31	21	10
		700	800	27	18	9
		800	900	22	15	7
		900	1,000	17	11	6

TABLE—*continued.*

Lift, feet.		Lead, feet.		Average Day's Work, cubic feet.		
From	To	From	To	Man.	Woman.	Child.
10	15	...	100	48	32	16
inclusive.		100	200	45	30	15
		200	300	41	27	14
		300	400	38	25	13
		400	500	34	23	11
		500	600	30	20	10
		600	700	27	18	9
		700	800	24	16	8
		800	900	20	13	7
		900	1,000	16	11	5
15	20	...	100	42	28	14
inclusive.		100	200	39	26	13
		200	300	36	24	12
		300	400	33	22	11
		400	500	30	20	10
		500	600	27	18	9
		600	700	24	16	8
		700	800	21	14	7
		800	900	18	12	6
		900	1,000	15	10	5
20	25	...	100	37	25	12
inclusive.		100	200	35	23	12
		200	300	32	21	11
		300	400	30	20	10
		400	500	27	18	9
		500	600	25	17	8
		600	700	22	15	7
		700	800	19	13	6
		800	900	17	11	6
		900	1,000	14	9	5
25	30	...	100	33	22	11
inclusive.		100	200	31	21	10
		200	300	29	19	10
		300	400	27	18	9
		400	500	25	17	8
		500	600	23	15	8
		600	700	21	14	7
		700	800	19	13	6
		800	900	16	11	5
		900	1,000	13	9	4

TABLE—*continued.*

Lift, feet.		Lead, feet.		Average Day's Work, cubic feet.		
From	To	From	To	Man.	Woman.	Child.
30 inclusive.	35	...	100	30	20	10
		100	200	28	19	9
		200	300	26	17	9
		300	400	24	16	8
		400	500	22	15	7
		500	600	20	13	7
		600	700	18	12	6
		700	800	16	11	5
		800	900	14	9	5
		900	1,000	12	8	4
35 inclusive.	40	...	100	28	19	9
		100	200	26	17	9
		200	300	24	16	8
		300	400	22	15	7
		400	500	20	13	7
		500	600	18	12	6
		600	700	16	11	5
		700	800	14	9	5
		800	900	13	9	4
		900	1,000	12	8	4
40 inclusive.	45	...	100	26	17	9
		100	200	24	16	8
		200	300	23	15	8
		300	400	21	14	7
		400	500	20	13	7
		500	600	18	12	6
		600	700	17	11	6
		700	800	15	10	5
		800	900	13	9	4
		900	1,000	11	7	4
45 inclusive.	50	...	100	24	16	8
		100	200	23	15	8
		200	300	21	14	7
		300	400	20	13	7
		400	500	18	12	6
		500	600	17	11	6
		600	700	15	10	5
		700	800	14	9	5
		800	900	12	8	4
		900	1,000	10	7	3

TABLE—*continued.*

lft, feet.		Lead, feet.		Average Day's Work, cubic feet.		
From	To	From	To	Man.	Woman.	Child.
50 inclusive.	55	...	100	22	15	7
		100	200	21	14	7
		200	300	20	13	7
		300	400	18	12	6
		400	500	16	11	5
		500	600	15	10	5
		600	700	14	9	5
		700	800	13	9	4
		800	900	12	8	4
		900	1,000	10	7	3
55 inclusive.	60	...	100	21	14	7
		100	200	20	13	7
		200	300	18	12	6
		300	400	17	11	6
		400	500	15	10	5
		500	600	14	9	5
		600	700	13	9	4
		700	800	12	8	4
		800	900	10	7	3
		900	1,000	9	6	3

The above quantities must be multiplied, for:—

Soft murum by	0·82	Excavation.
Average murum by	0·64	
Hard murum by	0·46	
Loose earth or sand by	1·25	Spoil.
Loose murum by	1·00	

APPENDIX 24.

TABLES[1] FOR ASCERTAINING THE COST OF CARRIAGE BY COOLIES AND BY CARTS.

Note on the Cost of Carriage.

When any quantity of materials of given weight or cubic content (L) has to be moved to any distance in feet (d) by a succession of trips of a number of units of conveyance, whose load in lbs. or cubic feet is (l), then the cost of transport of (L) will depend :—

Firstly, on X, or the proportion of (L) to (l) ; that is, the number of unit-loads, or trips of a unit of conveyance, that will be necessary.

Secondly, on Y, or the fraction of a working day, taken by a unit to make one trip to (d) and back.

Thirdly, on Z, or the hire of a unit of conveyance for a working day.

In fact, the cost of transport will $= X \times Y \times Z$.

Tables for all possible values of Z would have to be very numerous. The following Tables, however, give some useful values of Y for various leads, also of $X \times Y$ for a few given values of X or $\frac{L}{l}$.

The calculation of Y requires the following data :—

M, the number of minutes in a working day.

t, the time in minutes taken to load and unload a unit.

*s, the average speed in feet per minute of a unit.

If, also, for convenience of notation,

T, represent the time in minutes taken for a unit to make one trip to (d) and back,

N, be the number of such trips made in a working day,

then

$$T = \frac{2d}{s} + t. \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (1)$$

$$N = \frac{M}{T} = \frac{M}{\frac{2d}{s} + t}. \quad \ldots \quad \ldots \quad \ldots \quad (2)$$

$$Y = \frac{T}{M} = \frac{\frac{2d}{s} + t}{M} = \frac{1}{N}. \quad \ldots \quad \ldots \quad (3)$$

If T be observed as well as t, then from (1) :—

$$s = \frac{2d.}{T - t}. \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (4)$$

* Note.—s, the average speed, is the harmonic mean between the observed or known speeds s', s'', of the unit when loaded and unloaded respectively, *i.e.* $s = \frac{2\,s'\,s''}{s' + s''}$.

[1] Extracted from Marryat's "Specifications, Rates, and Notes on Work," 5th ed. pp. 2—4.

TABLE I.

When the unit of conveyance = a COOLY LOAD, or $l = \frac{1}{2}$ cubic foot.
DATA: M = 500 minutes; $t = \frac{1}{2}$ minute; $s = 200$ feet; d as per column 1.
Formulæ $T = \frac{2d}{s} + t$; $N = \frac{M}{T}$; $Y = \frac{T}{M}$; $X = \frac{L}{l}$; Z = daily hire of unit.

Cost of transport of quantity L to d = X × Y × Z.

I.	II.	III.	IV.	V.	I.	II.	III.	IV.	V.
Lead or distance in feet.	No. of trips to d and back in a day.	Time of a trip to d and back in minutes.	Fraction of a day taken to make one trip to d and back expressed in decimals.	When $l = \frac{1}{2}$ cft. and L = 100 cubic ft., X × Y = 200 Y.	Lead or distance in feet.	No. of trips to d and back in a day.	Time of a trip to d and back in minutes.	Fraction of a day taken to make one trip to d and back expressed in decimals.	When $l = \frac{1}{2}$ cft. and L = 100 cubic ft., X × Y = 200 Y.
d.	N.	T.	Y.	X × Y.	d.	N.	T.	Y.	X × Y.
20	714	0·70	·0014	0·28	475	95	5·25	·0105	2.10
30	625	0·80	·0016	0·32	500	91	5·50	·0110	2·20
40	556	0·90	·0018	0·36	525	87	5·75	·0115	2·30
50	500	1·00	·0020	0·40	550	83	6·00	·0120	2·40
60	455	1·10	·0022	0·44	575	80	6·25	·0125	2·50
70	417	1·20	·0024	0·48	600	77	6·50	·0130	2·60
80	385	1·30	·0026	0·52	625	72	6·75	.0135	2·70
90	357	1·40	·0028	0·56	650	71	7·00	·0140	2·80
100	333	1·50	·0030	0·60	675	69	7·25	·0145	2·90
125	286	1·75	·0035	0·70	700	67	7·50	·0150	3·00
150	250	2·00	·0040	0·80	750	63	8·00	·0160	3·20
175	222	2·25	·0045	0·90	800	59	8·50	·0170	3·40
200	200	2·50	·0050	1·00	850	56	9·00	·0180	3·60
225	182	2·75	·0055	1·10	900	53	9·50	·0190	3·80
250	167	3·00	·0060	1·20	950	50	10·00	·0200	4·00
275	154	3·25	·0065	1·30	1,000	48	10·50	·0210	4·20
300	143	3·50	·0070	1·40	1,050	45	11·00	·0220	4·40
325	133	3·75	·0075	1·50	1,100	43	11·50	·0230	4·60
350	125	4·00	·0080	1·60	1,150	42	12·00	·0240	4·80
375	118	4·25	·0085	1·70	1,200	40	12·50	·0250	5·00
400	111	4·50	·0090	1·80	1,250	38	13·00	·0260	5·20
425	105	4·75	·0095	1·90	1,300	37	13·50	·0270	5·40
450	100	5·00	·0100	2·00	1,320	36·5	13·75	·0275	5·50

N.B.—Quantity carried in a day is N unit-loads.

Cost of carrying one cooly load to d = Y × hire of cooly;

Cost of carrying 100 cubic feet of any materials is X × Y × hire of cooly.

EXAMPLE.

What would be the cost of removing 100 cubic feet to 600 feet if coolies cost 3 annas a day?—Ans. 2·6 × 3 = 7·8 annas.

TABLE II.

When the unit of conveyance = a CART LOAD or l has the values in Columns V to XII.

DATA: M = 540 minutes; t = 10 minutes; s = 100 feet; d as per column 1.

Formulæ $T = \frac{2d}{s} + t$; $N = \frac{M}{T}$; $Y = \frac{T}{M}$; $X = \frac{L}{l}$; Z = daily hire of unit.

Cost of transport of quantity L to d = X × Y × Z.

d.	N.	T.	Y	X × Y FOR THE UNDERMENTIONED VALUES OF L AND l.							
				L = 100 cubic feet.					L = 1 ton.		
Lead or distance in feet.	No. of trips to d and back in a day.	Time of a trip to d and back in minutes.	Fraction of a day taken to make one trip to d and back expressed in decimals.	l in cft.	l in cft.	l in cft.	l in cft.	l in cft.	l in cwt.	l in cwt.	l in cwt.
				6	7	8	9	10	8	9	10
I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.	XI.	XII.
400	30	18·0	0·0333	0·556	0·476	0·416	0·370	0·333	0·0825	0·0730	0·0666
431	29	18·6	0·0345	0·575	0·493	0·431	0·383	0·345	0·0863	0·0766	0·0690
482	28	19·6	0·0357	0·595	0·510	0·446	0·397	0·357	0·0895	0·0793	0·0714
500	27	20·0	0·0370	0·617	0·529	0·463	0·411	0·370	0·0925	0·0826	0·0740
540	26	20·8	0·0384	0·640	0·549	0·480	0·427	0·384	0·0960	0·0853	0·0768
580	25	21·6	0·0400	0·666	0·572	0·500	0·444	0·400	0·1000	0·0889	0·0800
625	24	22·5	0·0416	0·693	0·594	0·520	0·462	0·416	0·1040	0·0925	0·0832
675	23	23·5	0·0435	0·724	0·621	0·544	0·483	0·435	0·1087	0·0966	0·0870
730	22	24·5	0·0454	0·757	0·649	0·567	0·504	0·454	0·1135	0·1009	0·0908
785	21	25·7	0·0476	0·793	0·680	0·595	0·529	0·476	0·1202	0·1068	0·0952
850	20	27·0	0·0500	0·833	0·714	0·625	0·555	0·500	0·1250	0·1111	0·1000
921	19	28·4	0·0526	0·877	0·751	0·657	0·584	0·526	0·1315	0·1169	0·1052
1,000	18	30·0	0·0555	0·925	0·793	0·694	0·617	0·555	0·1389	0·1234	0·1111
1,088	17	31·8	0·0588	0·980	0·840	0·735	0·653	0·588	0·1470	0·1307	0·1176
1,188	16	33·7	0·0625	1·042	0·893	0·781	0·694	0·625	0·1560	0·1390	0·1250
1,300	15	36·0	0·0666	1·111	0·952	0·833	0·740	0·666	0·1666	0·1480	0·1330
1,428	14	38·5	0·0714	1·190	1·020	0·893	0·793	0·714	0·1787	0·1590	0·1430
1,575	13	41·5	0·0769	1·282	1·099	0·961	0·854	0·769	0·1925	0·1710	0·1540
1,750	12	45·0	0·0833	1·389	1·190	1·042	0·926	0·833	0·2080	0·1850	0·1660
1,955	11	49·0	0·0909	1·515	1·299	1·136	1·010	0·909	0·2280	0·2020	0.1820
2,200	10	54·0	0·1000	1·666	1·429	1·250	1·111	1·000	0·2500	0·2220	0·2000
2,500	9·0	60·0	0·1111	1·852	1·587	1·389	1·234	1·111	0·2770	0·2470	0·2220
2,675	8·5	63·5	0·1176	1·960	1·680	1·470	1·307	1·176	0·2940	0·2610	0·2350
2,875	8·0	67·5	0·1250	2·083	1·786	1·563	1·389	1·250	0·3130	0·2770	0·2500
3,100	7·5	72·0	0·1333	2·222	1·904	1·666	1·481	1·333	0·3330	0·2960	0·2660
3,355	7·0	77·0	0·1428	2·383	2·040	1·784	1·587	1·428	0·3560	0·3170	0·2850
3,653	6·5	83·0	0·1538	2·563	2·197	1·922	1·709	1·538	0·3840	0·3410	0·3070
4,000	6·0	90·0	0·1666	2·777	2·381	2·083	1·852	1·666	0·4170	0·3700	0·3330
4,425	5·5	98·5	0·1818	3·030	2·597	2·273	2·020	1·818	0·4540	0·4030	0·3630
4,900	5·0	108·0	0·2000	3·333	2·857	2·500	2·222	2·000	0·5000	0·4450	0·4000
5,500	4·5	120·0	0·2222	3·703	3·174	2·777	2·409	2·222	0·5550	0·4940	0·4440
6,200	4·0	135·0	0·2500	4·166	3·571	3·125	2·777	2·500	0·6250	0·5560	0·5000
7,200	3·5	154·0	0·2857	4·762	4·082	3·573	3·174	2·857	0·7140	0·6340	0·5710
8,500	3·0	180·0	0·3333	5·555	4·762	4·167	3·704	3·333	0·8330	0·7410	0·6660
10,300	2·5	216·0	0·4000	6·666	5·714	5·000	4·444	4·000	1·000	0·8890	0·8000
13,000	2·0	270·0	0·5000	8·333	7·143	6·250	5·555	5·000	1·250	1·111	1·000
17,500	1·5	360·0	0·6666	11·111	9·524	8·333	7·407	6·666	1·670	1·480	1·333
26,500	1·0	540·0	1·000	16·666	14·286	12·500	11·111	10·00	2·500	2·220	2·000
53,500	0·5	1,080·0	2·000	33·333	28·572	25·000	22·222	20·00	5·000	4·440	4·000

EXAMPLE.

The cost of carrying 100 cubic feet metal 2,200 feet, if a cart hired at 12 annas a day takes 8 cubic feet as its load,

= 1·25 × hire of cart per day = 1·25 × 12 = 15 annas.

APPENDIX 25.

USEFUL MEMORANDA.[1]

I. General.

1 cubic foot of water = 62·425 lbs. = 0·557 cwt. = 0·028 ton.
1 cubic inch of water = 0·03612 lb.
1 gallon of water = 10 lbs. = 0·16 cubic foot.
1 cubic foot of water = 6·24 gallons = say 6¼ gallons.
1 cwt. of water = 1·8 cubic foot = 11·2 gallons.
1 ton of water = 35·9 cubic feet = 224 gallons.
Inches of rainfall × 2,323,200 = cubic feet per square mile.
Inches of rainfall × 14½ = millions of gallons per square mile.
Inches of rainfall × 3,630 = cubic feet per acre.
1 inch run-off per hour per square mile = 645·33 cubic feet per second ;
= one cubic foot per acre per second, nearly.
12 cubic feet per second = 1,036,800 cubic feet per day ;
= 1 million cubic feet per day, nearly.
1 cubic foot per second = 31,536,000 cubic feet per year.
1 cubic foot per minute = 9,000 gallons a day.
1 cubic foot per second = 540,000 gallons a day.
= half a million gallons a day, nearly.
Number of seconds in 1 day = 86,400.
1 horse-power = 33,000 foot lbs. per min. = 550 foot. lbs. per sec.

$$\text{Horse-power} = 62{\cdot}4 \times \frac{\text{cubic feet falling per second}}{550} \times \left(\text{height of fall in feet.}\right)$$

1 horse power = approximately 8·814 cubic feet per second falling 1 foot.
Square feet in 1 acre = 43,560.
Square feet in 1 square mile = 27,878,400.
Acres in 1 square mile = 640.
Feet per second × 0·68 = miles per hour.

[1] Extracted chiefly from Molesworth's "Pocket Book of Engineering Formulæ," and Marryat's "Specifications, Rates and Notes on Work."

II. Pressure of Water.

P = Pressure in lbs. per square inch.

H = Head of water in feet.

$P = H \times 0{\cdot}4335$. $H = P \times 2{\cdot}307$.

Pressure in lbs. per square foot = 62·4 H.

Pressure of Water at Different Heads.

H = Head in feet.

P = Pressure in cwts. per square foot = 0·557 H.

p = Pressure in lbs. per square inch = 0·433 H.

H	P	p	H	P	p	H	P	p	H	P	p
1	0·56	0·43	7	3·90	3·03	18	10·03	7·81	50	27·85	21·70
2	1·11	0·87	8	4·45	3·47	20	11·14	8·68	60	33·42	26·04
3	1·67	1·30	9	5·01	3·91	25	13·92	10·85	70	38·99	30·38
4	2·23	1·73	10	5·57	4·34	30	16·71	13·02	80	44·56	34·72
5	2·78	2·17	12	6·68	5·20	35	19·49	15·19	90	50·13	39·06
6	3·34	2·60	15	8·36	6·51	40	22·28	17·36	100	55·70	43·35

III. Velocity of Water.

V = Theoretical velocity in feet per second.

g = Force of gravity = 32·2.

$2g = 64{\cdot}4$. $\sqrt{2g} = 8{\cdot}025$. $\frac{1}{2g} = 0{\cdot}0155$.

H = Head of water in feet.

$V = \sqrt{2g\,H} = 8{\cdot}025\ \sqrt{H}$.

$H = \frac{V^2}{2g} = 0{\cdot}0155\ V^2$.

Theoretical Velocity due to Different Heads in Feet per Second.

Head in Feet.	0	1	2	3	4	5	6	7	8	9
0	...	8·025	11·35	13·90	16·05	17·94	19·66	21·23	22·69	24·08
10	25·38	26·62	27·80	28·93	30·00	31·08	32·10	33·09	34·05	34·98
20	35·89	36·77	37·64	38·48	39·31	40·12	40·92	41·70	42·40	43·21
30	43·95	44·68	45·40	46·10	46·79	47·47	48·15	48·82	49·47	50·11
40	50·75	51·39	52·01	52·62	53·23	53·83	54·43	55·02	55·60	56·18
50	56·74	57·31	57·87	58·42	58·97	59·51	60·05	60·59	61·11	61·64
60	62·16	62·68	63·19	63·69	64·20	64·70	65·19	65·69	66·18	66·66
70	67·14	67·62	68·09	68·57	69·03	69·50	69·96	70·42	70·87	71·33
80	71·78	72·23	72·67	73·11	73·55	74·00	74·42	74·85	75·28	75·71
90	76·13	76·55	76·97	77·39	77·81	78·22	78·63	79·04	79·44	79·85

IV. Discharge of Water from Sluices.

V = Theoretical velocity in feet per second due to the head of water (from surface of water upstream to that downstream);

A = Area of aperture in square feet;

Q = Discharge in cubic feet per second;

E = Velocity of efflux in feet per second;

$E = V K$. $Q = E A = V K A$.

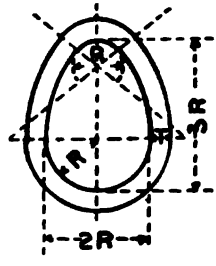
For large sluices with pointed piers ... $K = 0{\cdot}95$.

For small sluices $K = 0{\cdot}85$.

For pipe sluices, bore = $\frac{1}{4}$ to $\frac{1}{24}$ length $K = 0{\cdot}77$ to $0{\cdot}73$.

V. Areas of Outlet Culverts.

(Proportion of Height to Extreme Width Inside 3 to 2.)



Size of Culvert in Inches.	Internal Area of Culvert in Square Feet.	Size of Culvert in Inches.	Internal Area of Culvert in Square Feet.
36 × 54	10·350	56 × 84	25·044
38 × 57	11·532	58 × 87	26·865
40 × 60	12·778	60 × 90	28·750
42 × 63	14·087	62 × 93	30·699
44 × 66	15·461	64 × 96	32·711
46 × 69	16·898	66 × 99	34·788
48 × 72	18·400	68 × 102	36·928
50 × 75	19·965	70 × 105	39·132
52 × 78	21·594	72 × 108	41·400
54 × 81	23·287		

Thickness of Culvert Rings.

	Culverts under 48″ × 72″	Culverts 48″ × 72″ and over.
For culverts under dams less than 50 feet high	1′ 3″	1′ 9″
„ „ „ more „ 50 „	1′ 6″	2′ 0″

APPENDIX 26.

STATISTICAL INFORMATION REQUIRED FOR A PROJECT.

(*Vide* Chapter V., paragraph 242 (*a*).)

Consecutive No.	Items.	Unit.	Amount.
	A.—ENGINEERING.		
	I. General.		
1	Area of Catchment	sqr. miles	
2	Average annual rainfall	inches	
3	Estimated percentage of run-off to average rainfall	per cent.	
4	Estimated depth of run-off from average rainfall (2) × (3)	inches	
5	Estimated average annual yield from catchment (1) × (4)	mill. cub. ft.	
6	Percentage of average annual yield to available Full-Supply Storage capacity of reservoir (5) and (*II.*—5) . .	per cent.	
7	Percentage of number of years of ordinary minimum rainfall . . .	do.	
8	Ordinary minimum annual rainfall . .	inches	
9	Estimated percentage of run-off of (8) .	per cent.	
10	Estimated depth of run-off from ordinary minimum rainfall (8) × (9) . .	inches	
11	Estimated yield from ordinary minimum rainfall (1) × (10)	mill. cub. ft.	
12	Percentage of ordinary minimum yield to available Full-Supply Storage capacity of reservoir (11) and (*II.*—5) . .	per cent.	
13	Depth of run-off required to produce available Full-Supply Storage of reservoir (1) and (*II.*—5)	inches	
14	Average amount of fair-weather flow of impounded stream during . . .	November / December / February / April	

TABLE—*continued.*

Consecutive No.	Items.	Unit.	Amount
	A.—ENGINEERING—*continued.*		
	II. Tank.		
1	Area of Tank at Full-Supply level .	mill. sq. ft. acres	
2	Area of Tank at Outlet Sill level . .	do. do.	
3	Mean area of Tank ½ [(1) + (2)] . .	do. do.	
4	Total Full-Supply Storage capacity .	mill. cub. ft.	
5	Gross available Full-Supply Storage capacity above Outlet Sill . . .	do. do.	
6	Loss of storage by evaporation and absorption (3) × (9)	do. do.	
7	Net Full-Supply Storage available for irrigation (5) – (6)	do. do.	
8	Average depth of available Full-Supply Storage (5) ÷ (3)	feet	
9	Estimated depth lost by evaporation and absorption on mean area of Tank (3) .	do.	
10	Fall of River bed in Tank Full-Supply limits	feet per mile	
	III. Dam.		
1	Length of Dam at top	feet	
2	Maximum height of Dam above ground level	do.	
3	Top of Dam	R.L.	
4	High-Flood level	do.	
5	Full-Supply level	do.	
6	Outlet Sill level	do.	
7	Lowest foundation level of Dam . .	do.	
8	Top-width of Dam	feet	
9	Upstream slope of Dam	ratio	
10	Downstream slope of Dam . . .	do.	
11	Maximum depth of Puddle Trench . .	feet	
12	Bottom-width of Puddle Trench . .	do.	
13	Side-slopes of Puddle Trench . . .	ratio	
14	Top of Pitching	R.L.	
15	Bottom of Pitching	do.	
16	Thickness of Pitching	feet	
17	Maximum depth of Full-Supply (5) – (7)	do.	
18	Depth from Full-Supply to Outlet Sill (5) – (6)	do.	

TABLE—*continued.*

Consecutive No.	Items.	Unit.	Amount.
	A.—ENGINEERING—*continued.*		
	IV. Waste-Weir.		
1	Description of Waste-Weir (Clear overfall, drowned or channel) . . .	—	
2	Total length of Waste-Weir . . .	feet	
3	Net length of overfall of Weir Crest .	do.	
4	Under-sluices	No. and size	
5	Automatic Gates	do. do.	
6	Calculated High-Flood depth over Waste-Weir Crest	feet	
7	Calculated High-Flood discharge of Waste-Weir Crest	cusecs.	
8	Do. do. do. of Under-sluices	do.	
9	Do. do. do. of Automatic Gates.	do.	
10	Do. do. do. of Whole Weir (7) + (8) + (9)	do.	
11	Estimated High-Flood discharge from Catchment	do.	
12	Calculated rate of run-off from Catchment of Waste-Weir High-Flood discharge	in. per hour	
13	Estimated rate of maximum run-off from Catchment	do. do.	
	V. Outlet.		
1	Description of Outlet (Culvert under Dam, Headwall, &c.)	—	
2	Sluices	No. and size	
3	Discharging capacity of Sluices at High-Flood level	cusecs.	
4	Do. do. do. at Full-Supply level	do.	
5	Do. do. do. one foot above top of Sluices	do.	
	VI. Canal.		
1	Description of Headworks (Tank Outlet, Weir, &c.)	—	
2	Distance of Headworks from Tank . .	miles	
3	Estimated loss in transit . . .	per cent.	
4	Length of Main Canal	miles	
5	Length of Distributaries	do.	

TABLE—*continued.*

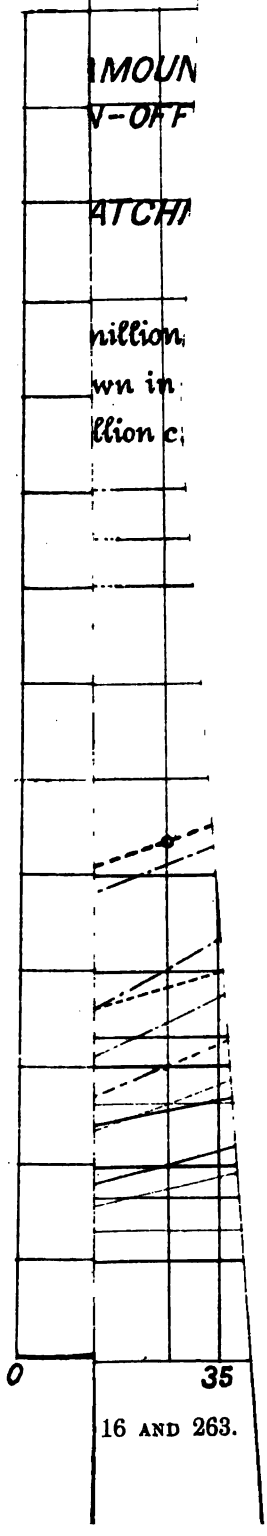
Consecutive No.	Items.	Unit.	Amount.
	A.—ENGINEERING—*continued.*		
	VI. Canal—continued.		
6	Full-Supply discharge of Canal . .	cusecs.	
7	Estimated duty of water at Canal Head .	acres per cusec.	
	VII. Land.	Tank. / Canal.	Total.
1	Total area to be acquired in acres		
2	Culturable area do. do.		
3	Percentage of (2) to (4)	per cent.	
4	Area irrigable by project in average years	acres	
5	Do. do. do. in years of minimum rainfall	do.	
6	Culturable area commanded by project .	do.	
7	Percentage of (4) to (6)	per cent.	
8	Percentage of (5) to (6)	do.	
	B.—FINANCIAL.		
1	Estimated cost of Dam	Rs.	
2	Estimated cost of Waste-Weir . .	,,	
3	Estimated cost of Outlet . . .	,,	
4	Estimated cost of Land Compensation .	,,	
5	Estimated cost of Buildings and Miscellaneous Expenditure	,,	
6	Total estimated cost of Tank . . .	,,	
7	Estimated cost of Canal	,,	
8	Total estimated cost of Project for Works (6) + (7)	,,	
9	Establishment at 23 per cent. on (8) less cost of excluded items (Rs.) . .	,,	
10	Tools and Plant at 1½ per cent. on (8) less do. do.	,,	
11	Total Direct Charges (8) + (9) + (10) .	,,	
12	Capitalisation of abatement of Land Revenue	,,	
13	Leave and Pension allowances at 14 per cent. on (9)	,,	
14	Total Indirect Charges (12) + (13) . .	,,	
15	Grand Total, Direct and Indirect Charges (11) + (14)	,,	

TABLE—*continued.*

Consecutive No.	Items.	Unit.	Amount.
	B.—FINANCIAL—*continued*		
16	Area irrigable	acres	
17	Gross Revenue at Rs. per acre . .	Rs.	
18	Deduct working expenses at Rs. per acre	,,	
19	Net Revenue (17)—(18)	,,	
20	Percentage return of (19) on (15) . .	per cent.	
21	Estimated rate of gross available Full-Supply Storage (6) ÷ (*II.*—5) . .	Rs. per mill. cub.ft.	
22	Estimated rate of Direct Charges per acre irrigable by Project (11) ÷ (16)	Rs.	
23	Estimated rate of Direct and Indirect Charges per acre irrigable by Project (15) ÷ (16).	,,	
24	Normal value of work available for Famine Relief	,,	
25	Relief Labourers for whom employment can be found on Works for 6 months	No.	

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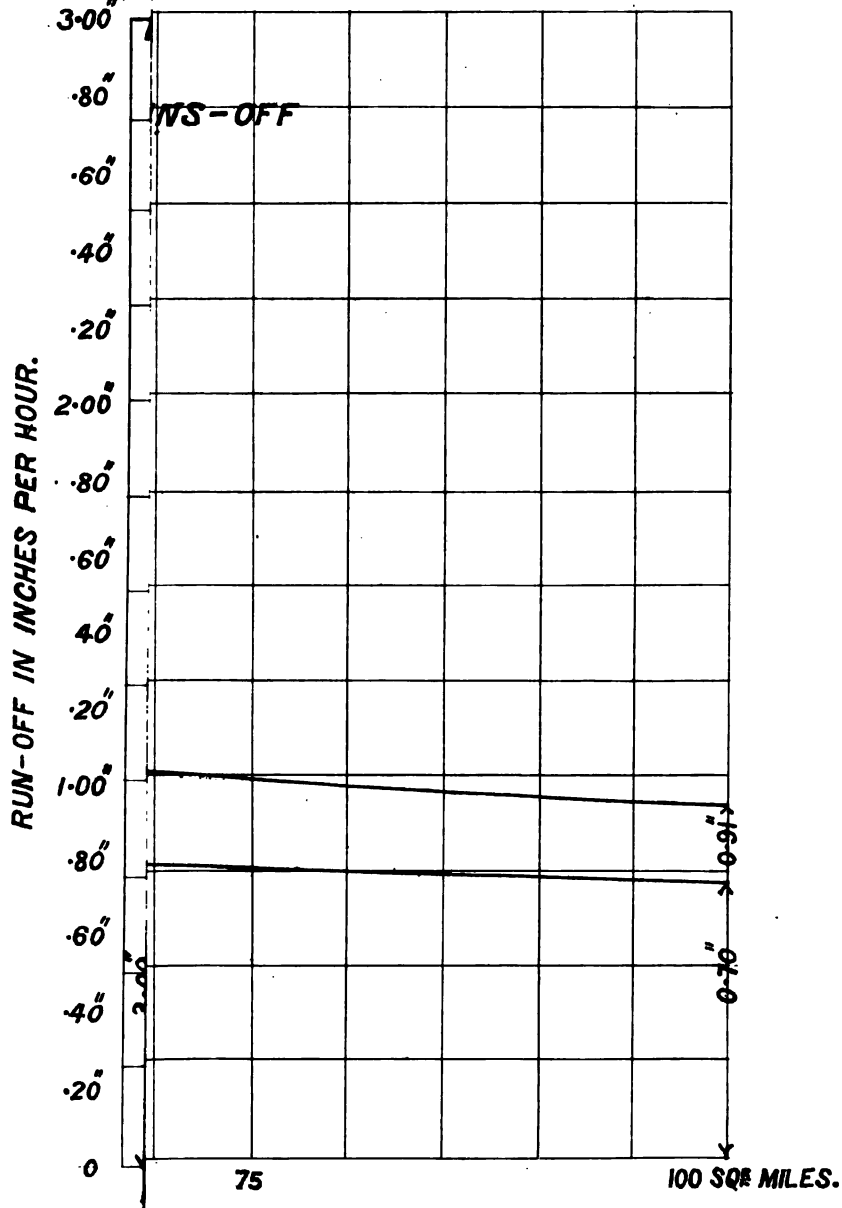
BRADBURY, AGNEW, & CO. LD., PRINTERS, LONDON AND TONBRIDGE.



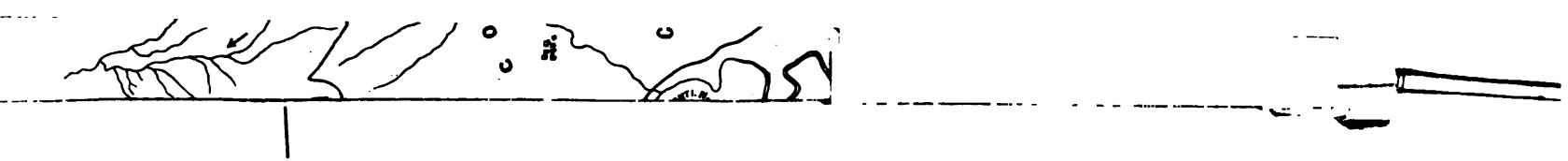
16 AND 263.

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PLATE Nº 2.

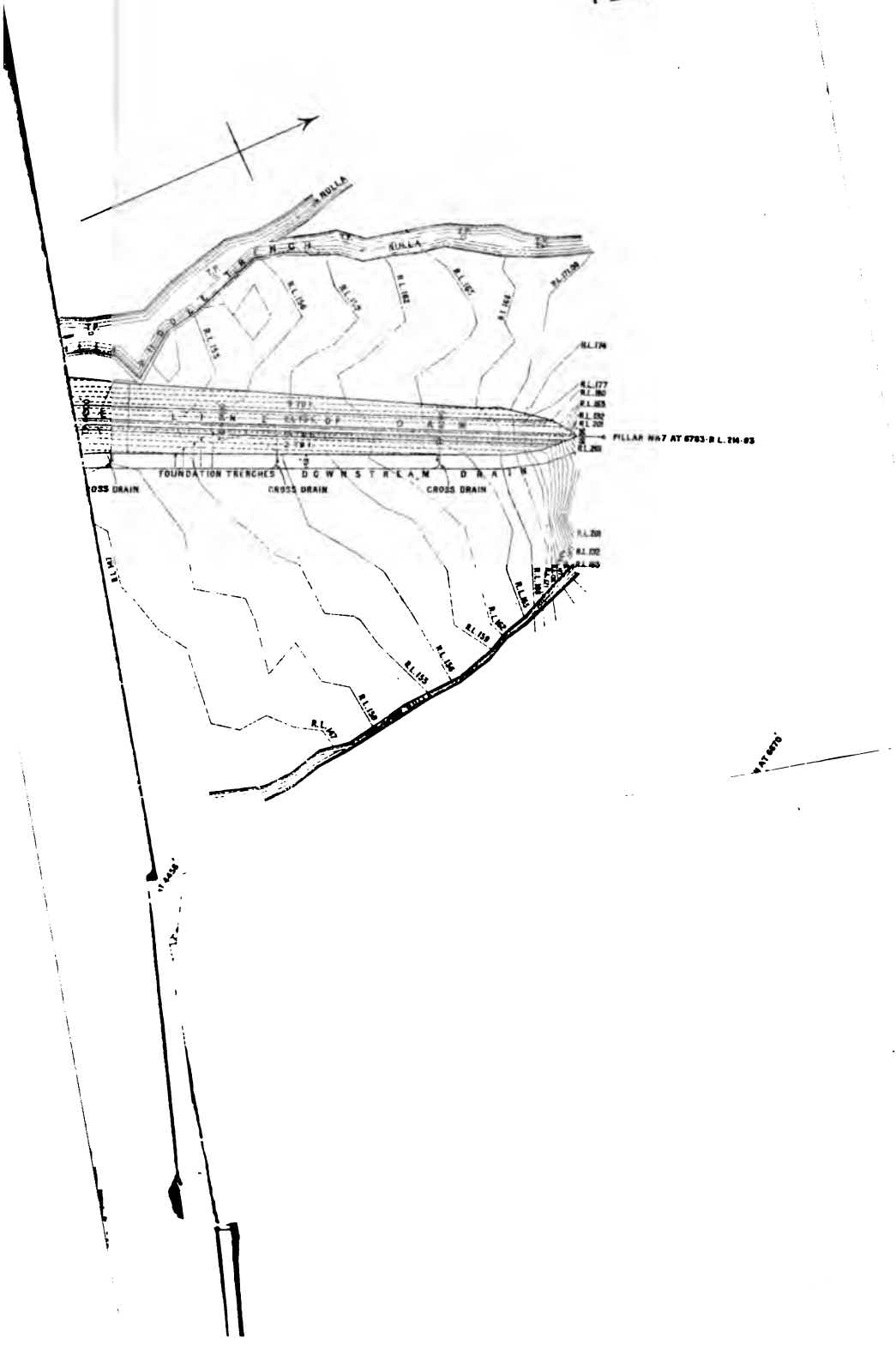

3·00"
·80"
·60"
·40"
·20"
2·00"
·80"
·60"
40"
·20"
1·00"
·80"
·60"
·40"
·20"
0
RUN-OFF IN INCHES PER HOUR.
75
100 SQE MILES.
0·91"
0·70"


OF THE
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OF

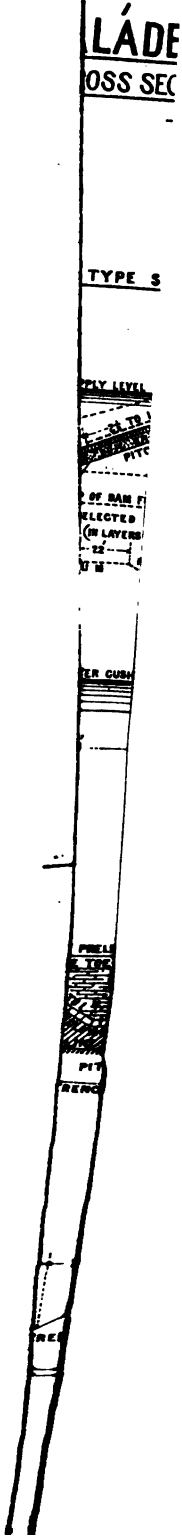


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PLATE Nº 4.


KULLA
KULLA
R.L.176
R.L.177
R.L.165
R.L.152
R.L.201
PILLAR No7 AT 6783·8 L. 214·03
FOUNDATION TRENCHES
DOWNSTREAM DRAIN
CROSS DRAIN
CROSS DRAIN
CROSS DRAIN
R.L.201
R.L.192
R.L.185
R.L.190
R.L.182
R.L.159
R.L.155
R.L.154
R.L.150
R.L.147
AT 6870



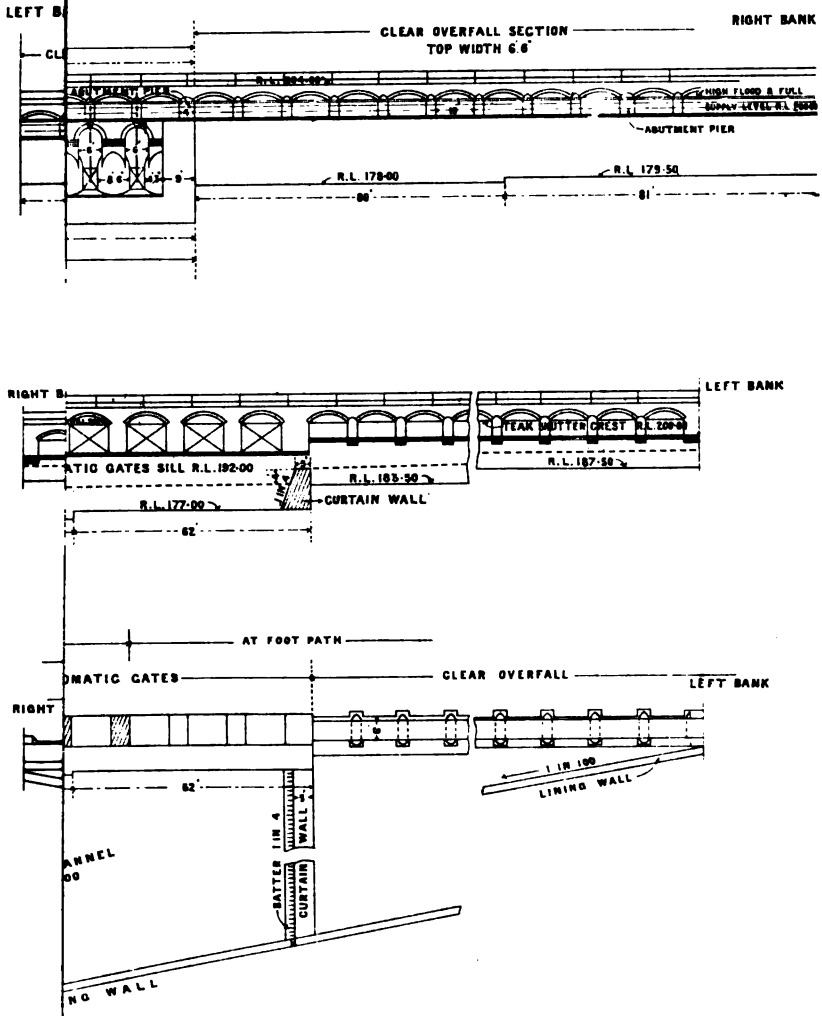


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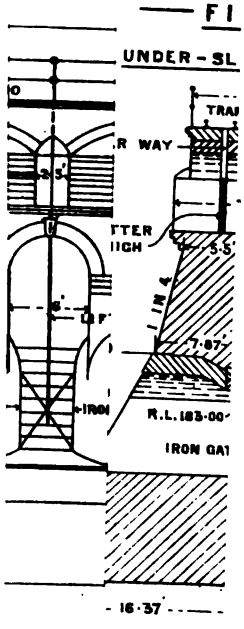
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PLATE Nº 7.


CLEAR OVERFALL SECTION
TOP WIDTH 6.6'
RIGHT BANK
ABUTMENT PIER
ABUTMENT PIER
R.L. 178-00
R.L. 179-50
80'
81'
LEFT BANK
PEAK SPILLWAY CREST R.L. 200-00
R.L. 197-50
R.L. 193-50
CURTAIN WALL
R.L. 177-00
62'
AT FOOT PATH
CLEAR OVERFALL
LEFT BANK
62'
1 IN 100
LINING WALL
WALL
CURTAIN

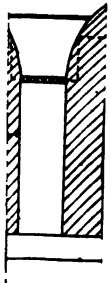

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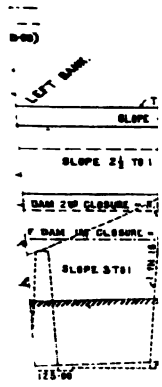
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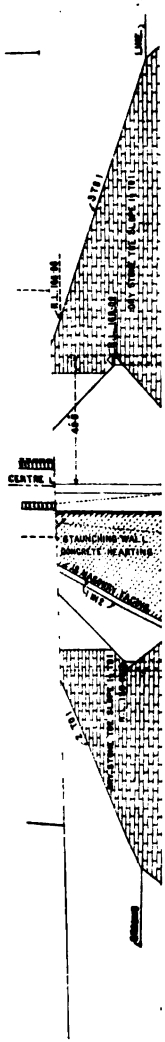
S FOR NEED




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CT


LEFT BANK
SLOPE
SLOPE 2½ TO 1
DAM 2ND CLOSURE
DAM 1ST CLOSURE
SLOPE 3 TO 1


CENTRE
STAUNCHING WALL
CONCRETE HEARTING

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FIG. 3. SECTION ON A. B.

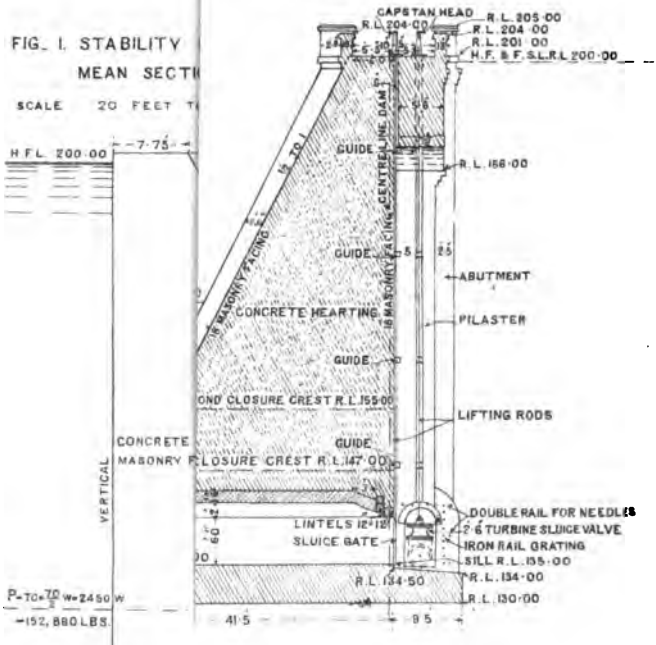
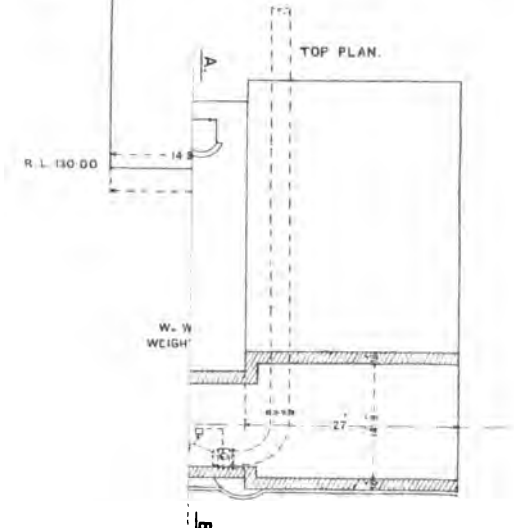


FIG. 1. STABILITY
MEAN SECTI

SCALE 20 FEET T



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E.


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SCALE — $\frac{3}{4}$" = 1 FOOT.

FIG. 4.

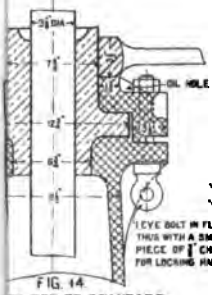
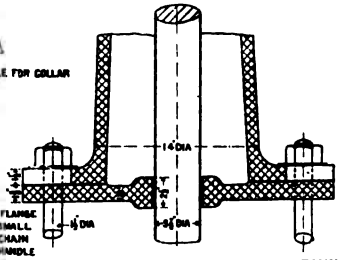


FIG. 5.



SCALE — $\frac{3}{4}$" = 1 FOOT

FIG. 14

OF TOP OF STANDARD.

FIG. 6.

SECTION AT G. G.

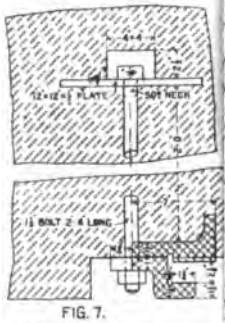


FIG. 16.

JOINT TO ROD.

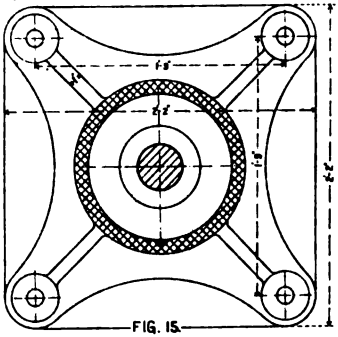


FIG. 15.

DETAIL OF BASE OF STANDARD

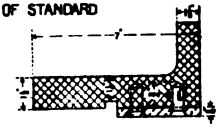


FIG. 7.

SECTION AT E. E.



FIG. 17.

SECTION THRO JOINT

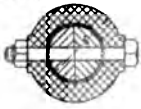


FIG. 8.

DETAL OF STEEL

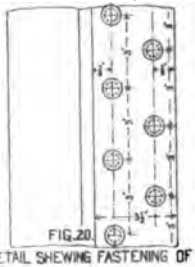


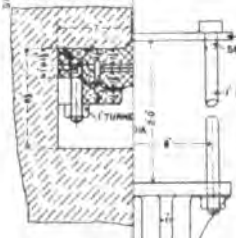
FIG. 20.

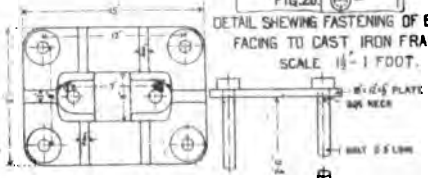
DETAIL SHEWING FASTENING OF BRASS FACING TO CAST IRON FRAME.

SCALE 1½" = 1 FOOT.

FIG. 9.

SECTION AT





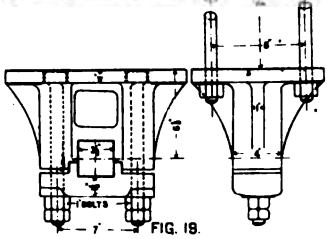


FIG. 18.

DETAIL OF GUIDE TO SQUARE ROD.

D ROD.


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PLATE Nº 12.

FIG_4. PLAN OF CAPSTAN HEADS

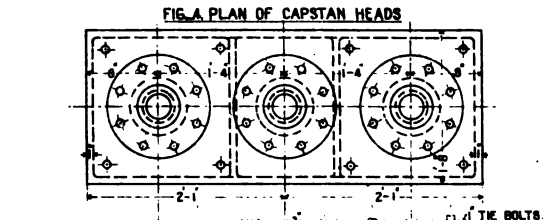


FIG_5. PLAN OF CIRCULAR GUIDE BLOCKS.

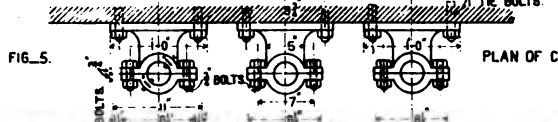
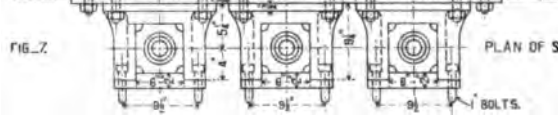


FIG_6. PLAN OF BRASS FEMALE SCREW BLOCKS.





FIG_7. PLAN OF SQUARE GUIDE BLOCKS.



BATTMAN'S CAPSTAN HEAD

SCALE—$\frac{3}{4}$ INCHE TO 1 FOOT

FIG_8. SECTION AT E. F.

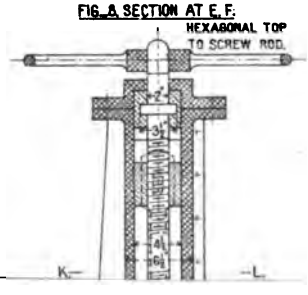
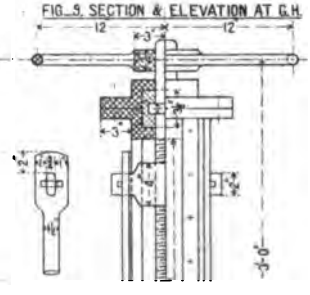
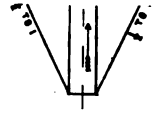


FIG_9. SECTION & ELEVATION AT G.H.

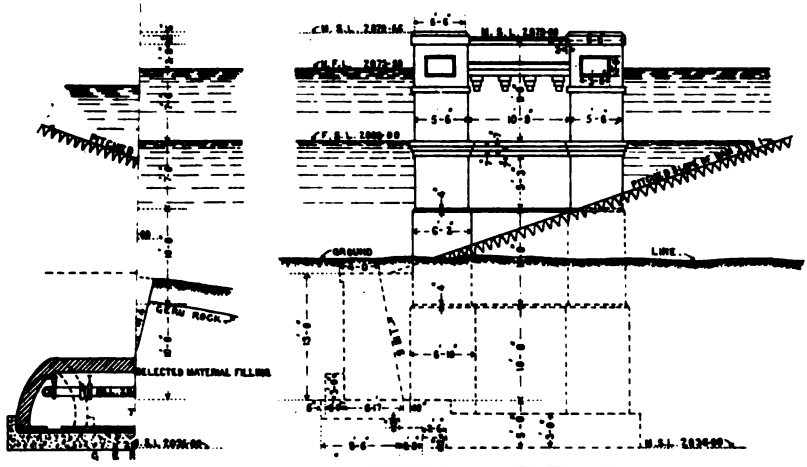


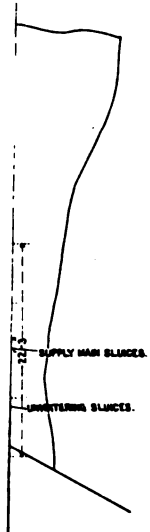


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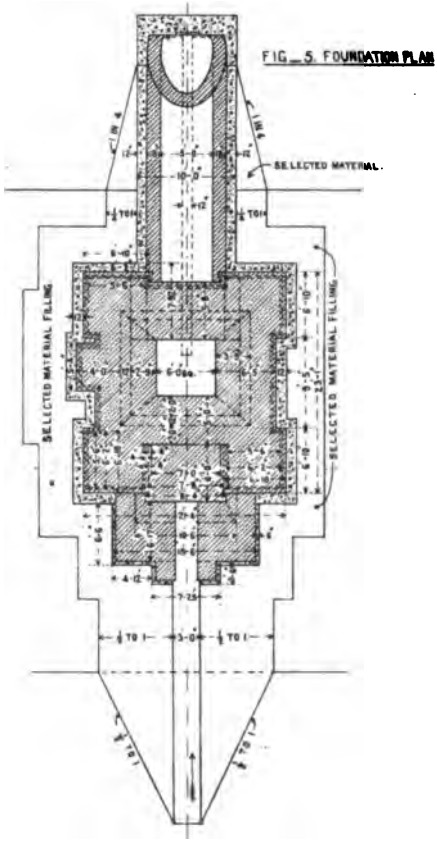
PLATE Nº 13.

FIG—3. SIDE ELEVATION

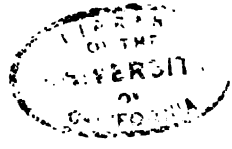




FIG—5. FOUNDATION PLAN




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